ELDEAN BRIDGE
(Allen's Mill Bridge)
Spanning Great Miami River at bypassed section of Eldean Road
Troy vicinity
Miami County
Ohio

HAER OH-122
OH-122

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FIELD RECORDS

HISTORIC AMERICAN ENGINEERING RECORD
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HISTORIC AMERICAN ENGINEERING RECORD

ELDEAN BRIDGE
(Allen's Mill Bridge)

HAER No. OH-122

LOCATION:	Spanning Great Miami River at bypassed section of Eldean Road, Troy vicinity, Miami County, Ohio UTM: 16.737345.4440124 Troy, Ohio Quadrangle
STRUCTURAL TYPE:	Long through truss covered bridge
DATE OF CONSTRUCTION:	1860
DESIGNER/ BUILDER:	James and William Hamilton, Piqua, Ohio
OWNER:	Miami County, Ohio
PREVIOUS USE:	Vehicular and pedestrian bridge
PRESENT USE:	Vehicular and pedestrian bridge; bypassed in 1964
SIGNIFICANCE:	Eldean Bridge is one of the best surviving examples of a Long truss, patented by Col. Stephen H. Long in 1830. Long was the first American engineer to apply the concept of driving wedges into the connections to prestress the structure, ensuring that the members functioned efficiently.
AUTHORS:	Lola Bennett, Historian, Summer 2002 Francesca da Porta, HAER Engineering Technician, and Dario A. Gasparini, Ph.D., Professor of Civil Engineering, Case Western Reserve University, Summer 2002
PROJECT INFORMATION:	The National Covered Bridges Recording Project is part of the Historic American Engineering Record (HAER), a long-range program to document historically significant engineering and industrial works in the United States. HAER is administered by the Historic American Buildings Survey/Historic American Engineering Record, a division of the National Park Service, U.S. Department of the Interior. The Federal Highway Administration funded the project.

Chronology

1805 America's first covered bridge built at Philadelphia.

1809 Ohio's first covered bridge built.

1830 Col. Stephen H. Long patents Long truss.

1842 First bridge built at or near this site.

1848 Henry Ware Allen established grist and saw mill near this site.

1860 James and William Hamilton build Eldean Bridge.

1922 Original floor overlaid with green elm flooring.

1936 Metal tie rods added to trusses; stone pier and abutments capped with concrete.

1940s Metal roof laid over original wood shingle roof.

1950s Eldean Bridge posted for 4-ton weight limit.

1963 Eldean Bridge bypassed and closed to traffic.

1967 Eldean Bridge painted red.

1975 Eldean Bridge listed on the National Register of Historic Places.

1976 Eldean Bridge rehabilitated and opened to traffic.

1980 Eldean Bridge rehabilitated.

2002 Eldean Bridge recorded by the Historic American Engineering Record.

Introduction

The first documented covered bridge in Ohio was built in 1809 across Beaver Creek in Columbiana County.[1] Historians estimate that there were once as many as 4,000 covered bridges in Ohio, more than any other state in the nation.[2] Over time, these structures were lost to fire, floods, neglect and replacement. A list published by the state in 1937 indicates that, at that date, there were 609 covered bridges in Ohio. That number dropped dramatically in the 1950s as modern spans replaced covered bridges. Today, Ohio has 135 surviving covered bridges.

Miami County's first covered bridge was built across the Miami River on the Piqua-Urbana Turnpike in 1838.[3] Miami County Board of Commissioners Records indicate that the county was building a variety of stone, iron and wooden bridges in the 1850s and 1860s, with many of the major spans being wooden covered bridges. At least fourteen covered bridges were built across the Great Miami River and fifteen across the Stillwater River in Miami County in the mid-nineteenth century.[4] Of these, only the Eldean Bridge is still standing. According to Miriam Wood, by the 1870s, most of the major bridges had been constructed, and "Bridge contracts after the mid-1870s were almost all for iron trusses."[5] The Eldean Bridge is the second longest covered bridge in Ohio, and an excellent example of a Long truss.[6]

Description

Eldean Bridge is a two-span Long truss covered bridge on a mortared stone pier and abutments. The total length of the bridge is 218'-10" along the lower chord, each span measuring 108'-2".[7] The truss is 15'-10" high from the top of the upper chord to the bottom of the lower chord and 20'-9½" wide overall, with a roadway width of 16'-10".

The truss is white pine framed in the manner patented by Col. Stephen Harriman Long in 1830. The upper chord consists of three parallel lines of timbers (a thick 9½x12" plank, flanked by thinner 4x9½" planks) with wooden shear blocks between them, spliced at intervals between panel points. This assembly is fastened together with ¾" diameter threaded bolts, which pass through the chords and are fastened with a nut on the inner face. The lower chord is four parallel lines of 5x11½" timbers with wooden shear blocks, bolted together in a similar manner. Timber posts connect the upper and lower chords and form twelve panels in each truss. There are paired timber braces and single counter

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[1] Miriam Wood, *The Covered Bridges of Ohio, an Atlas and History* (Columbus: Old Trail Printing Company, 1993), p.119.

[2] In 1970, covered bridge historian Richard Sanders Allen published a conservative estimate of 2,000 covered bridges. More recent historical research has doubled that estimate.

[3] Irene E. Miller, *History of Miami County, Ohio* (Tipp City: Miami County Historical Society, 1982), p.33.

[4] Wood, p.139.

[5] Ibid.

[6] The longest is the 236' Harpersfield Bridge over the Grand River in Ashtabula County.

[7] Measurements taken by National Covered Bridge Recording Project team members in summer 2002. The reported length of 234'-10" in various contemporary sources presumably refers to the overall length of the bridge at the ridgeline.

braces between the posts. The dimensions of posts and braces decrease from the end of the trusses to the center of each span, proportioned to take into account differing member forces, thereby equalizing member stresses and producing an efficient structure. The posts pass through the lower chord where they are set into notches and fastened with ¾" diameter bolts. There are wooden wedges (averaging about 1½' long, and tapering from ¾" at the point to 1¼" at the butt) in the joints between the bottom of the counterbraces and the posts. As prescribed in Colonel Long's patent, these wedges were used to prestress the truss when it was built, but the connections have loosened over time and the majority of the wedges no longer function.[8] In 1936, paired vertical tie rods were added on either side of a number of posts in both trusses because of broken post shoulders. A metal plate and nut assembly secures the rods at the upper side of upper chord and the bottom side of the lower chord. Three diagonal tie rods were added at the southwest corner of the easterly truss in 1980 because of a broken lower chord. Steel I-beams were installed under the lower chords at the ends of the trusses in 1976 and under the lower chords at the center pier in 1980, effectively reducing the clear span of the trusses and the forces generated in them.

The floor system is composed of 6x12" wooden floor beams placed transversely at each panel point. The beams rest on the lower chord and the lower lateral bracing is fastened between them. Seven lines of wooden joists, or stringers, are laid on top of the floor beams and support the wood plank deck (diagonally laid elm flooring over an earlier transverse wood plank deck). There are diagonal wooden braces at the abutments and pier.

Roof framing consists of tie beams that rest on the upper chord and are framed to the rafters, which, in turn, support the roof. There is upper lateral bracing between the tie beams. The corrugated metal gable roof has slightly overhanging eaves. The metal roofing covers an earlier layer of wooden shingles. The layers are fastened to wooden purlins on top of the rafters. There are sway braces between the posts and tie beams at the ends of the trusses and midspan.

Vertical board siding covers the exterior of the bridge to about 1-½' below the upper chord. The sheathing is fastened to wooden nailers on the outer face of the truss. Windows have been cut and framed into the siding, two per span, on each side of the bridge. Wooden kick boards have been nailed longitudinally along the exterior at window level and roadway level. The portals are straight with curved openings and suggested pilasters; additionally, there are three-quarter height shelter panels on the inner face of the end panels. Painted wooden signs with the bridge name, construction date and builder's name have been attached in the tympanum above each portal.

The abutments and pier are cut, squared stone with mortared joints. The pier has a cutwater on its upstream side. The tops of both the pier and abutments have been capped with concrete, and the bottoms protected from scour by poured concrete fenders. The

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[8] According to a recent inspection of the bridge, 75 percent of these wedges were loose. See Appendix A, Engineering Report.

lower chords of the bridge rest on bedding timbers on top of the facewall. The backwall above the abutment and behind the bedding timbers serves as a retainer for the roadbed. Stone wingwalls extend from the backwall at the westerly end of the bridge where the roadway approach has been graded on an incline.

History

The Eldean Bridge, originally known as the Allen's Mill Bridge, derives its name from two sites just west of its location: Allen's Mill, a mid-nineteenth to early-twentieth century flour mill, and Eldean Station, a stop on the Baltimore & Ohio Railroad branch line between Troy and Piqua. Shortly after the completion of the Miami and Erie Canal in the early 1840s, Henry Gerard erected a saw and grist mill on the Great Miami River near this site. In 1848, Henry Ware Allen of Pembroke, Massachusetts purchased the mill, and over the next fifty years developed it into "the largest and the most progressive in all western Ohio," shipping flour to the eastern seaboard and Europe.[9] Allen later went into business with a Mr. Wheeler. The 1894 Miami County atlas shows Allen & Wheeler's Mills (by 1911, Allen & Wheeler Flour Mill Company) adjacent to Eldean Station just north of the bridge site.

Miami County road records indicate that there was a bridge at this site by 1842. A sketch within these records shows the "road to Dayton" crossing Miami River at a bridge next to an old ford, but no other documentation has been found concerning that bridge.[10]

On March 7, 1860, the Miami County Board of Commissioners met and agreed to re-build the bridge "across the Miami River at H.W. Allen's Mill, to be built on Long's plan with stone abutments and pier."[11] The county auditor advertised the letting of the contracts in the *Troy Times* and *Piqua Enquirer* for three weeks. On April 13, 1860, James and William Hamilton won the contract for the bridge at Allen's Mill.

The Hamiltons built the bridge in the summer of 1860, completing it in the fall. The commissioners inspected it on December 11. The contractors received $1,337.68 for building the pier and abutments; $2,632.00 for the superstructure; and an additional $111.15 for completing the wing walls in 1861, for a total of $4,080.83.[12]

By the 1950s, the Eldean Bridge was badly deteriorated and posted for a 4-ton weight limit. Rather than replacing the bridge, the county decided to preserve it as an historic structure, bypassing a short section of Eldean Road, and constructing a new three-span steel beam bridge 125' north of covered bridge. In March 1964, the Miami County engineer reported to the Southern Ohio Covered Bridge Association: "We are in the

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[9] Miami Sesquicentennial Committee, *A History of Miami County, Ohio, 1807-1953* (Columbus: F.S. Heer Printing Co., 1953), p.165.
[10] Miami County Road Records, Volume 3, p.239.
[11] Miami County Board of Commissioners Records, *Journal V* (1848-1861), p.344.
[12] Miami County Board of Commissioners Records, *Journal V* (1848-1861).

process of constructing a new bridge beside the covered bridge to be completed by the middle of June."[13] A parking area and picnic shelter were built adjacent to the bridge.

By assigning the bypassed section of road (now Farver Road) a township road number, the county was able to keep the bridge listed in their inventory and maintenance records and have it regularly inspected. In 1967 missing sideboards were replaced and the siding painted red.[14] This siding was repeatedly vandalized to create openings in the sides of the bridge for fishing, so in 1976, the county engineers came up with two simple but effective solutions. First, they jacked the bottom chords and installed new cross bracing to allow the bridge to be reopened to limited traffic. Second, they cut and framed windows in the siding and installed kickboards on the outside of the bridge to prevent the siding from being removed. This has greatly reduced vandalism to the structure.

In 1980 the bridge was once again closed due to structural deterioration. Additional support was added over the center pier and three diagonal threaded rods installed from the lower chord to the upper chord to help transfer loads. The bridge reopened to traffic in June with posted weight limits. Today the Miami County Engineers Office inspects the bridge annually and maintains it. Many local people continue to enjoy this as a favorite recreation spot for picnicking and fishing, and covered bridge enthusiasts are frequent visitors.

The Eldean Bridge was listed on the National Register of Historic Places in 1975.

Builders
Brothers James and William Hamilton were stonemasons and quarry owners from Piqua. Their stone quarry, now known as Armco Quarry, was located at the southern edge of Piqua. According to Miami County Board of Commissioners Records, in 1858 they were awarded at least three major contracts for bridge masonry in Miami County. In 1860 they received two contracts for wooden bridges in Miami County: a 224' Long truss over Great Miami River (the Eldean Bridge) and a 95' Long truss bridge over Greenville Creek.[15]

[13] Southern Ohio Covered Bridge Association, *Covered Bridge Chatter* 2 (July 1964).
[14] *Piqua Daily Call*, August 7, 1967. Pre-1967 photos show that the bridge siding was formerly natural weathered gray.
[15] According to historian David Simmons, the Hamiltons may have acted as general contractors for the construction of the Eldean Bridge, subletting the contract for the timber superstructure to a local bridge builder, such as Robert W. Smith. James Hamilton was one of the early partners in Robert W. Smith's bridge business, and James's brother, John, later moved to Michigan where he became secretary of Smith's company. The professional associations of the Hamiltons and Smith may have begun with the Eldean Bridge. While no conclusive documentary evidence has been found concerning this, the Miami County Commissioners Records indicate that county bridge contracts were commonly split between masonry and superstructure work, and that the Hamilton brothers were generally awarded contracts for masonry only.

Design

Col. Stephen Harriman Long was born at Hopkinton, New Hampshire, on December 30, 1784. He attended Dartmouth College, taught mathematics at West Point, and became an army engineer in 1814. From 1816 to 1827 he undertook numerous surveys for the U.S. Army Topographical Engineers. He surveyed sites for canals, explored the upper Mississippi River and led expeditions in the West. In 1827 the War Department assigned him to be a consulting engineer for the Baltimore & Ohio Railroad. It was during this time that Long became interested in the design and construction of bridges.

In 1830 he obtained a patent for a wooden truss bridge with diagonal compression members and vertical tension members, and received patents for variations on this design in 1836 and 1839. Central to Long's patented design was the concept of driving wedges into the connections, which prestressed the structure, ensuring that the compression and tension members functioned efficiently.

The Long truss is considered one of the first scientifically designed bridge truss types because the forces in the members were determined mathematically, unlike earlier bridge trusses, which had been designed by empirical methods. Early nineteenth century French engineer Claude-Louis Navier (1785-1836) noted that a parallel chord truss can be treated as a beam with a stiffness proportional to the area of the chords times the square of the distance between them. By using this analogy, Long was able to determine the required chord areas in the truss.[16] The Long truss was a forerunner of iron panel-type trusses and one of the first to incorporate economy of design through the use of continuous framing over the piers.[17]

Because of its ability to carry heavy loads without excessive deflection, Long's design was used for the first truss railroad bridges built in the United States in 1831-32. The advent of the Howe truss in 1840, which was essentially a Long truss with iron verticals instead of wooden ones, eclipsed the widespread popularity of the Long truss.[18]

The design of the Eldean Bridge is consistent with Colonel Long's patent, except the connections of the diagonal members have been simplified, the prestressing wedges only appear at the bottom of the counterbraces, and there are no counterbraces in the end panels. The Eldean Bridge is one of the longest and most intact examples of Long's patented bridge truss in the United States.

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[16] D.A. Gasparini and C. Provost, "Contributions of Stephen Harriman Long to the Design of Trusses," CSCE Centennial Conference, Montreal, 1987, p.809

[17] John Diehl, "Bridge to the Past," *Timeline* 15 (May/June 1998): 36.

[18] According to D.A. Gasparini and C. Provost's 1987 paper, "Contributions of Stephen Harriman Long to the Design of Trusses": "[Long's] two bridge patents of 1830 and 1839 were the bases for the very successful Howe and Pratt trusses."

APPENDIX A: ENGINEERING REPORT

Introduction

This analysis is a structural study of the static behavior of the Eldean Bridge, built with the Long truss, which is significant because it does not use supplementary arches, as in a Burr arch-truss bridge, but rather relies completely on truss action for its structural strength. Long designed his truss using the principles and models of Claude-Louis Navier, and he introduced prestressing by driving wedges, or "keys," between members at joints to improve the truss' stiffness.

A principal objective of this study was to quantify the effect of these wedges in order to evaluate the effectiveness of Long's prestressing technique. Additional objectives were to quantify the static behavior of the Eldean Bridge under deal load and live load, determine the effects of shrinkage and creep on the structure, and provide guidance for assessment, rehabilitation, and maintenance of this type of truss.

Scope of Study

The main features of Long's trusses, and of the Eldean Bridge in particular, were determined from available documents and on-site measurements. A review of wood properties, in particular the temporal properties of shrinkage and creep, is presented. Common mechanical and physical property values were used for structural analyses of the bridge. Specific information on the temporal properties of wood was gathered in order to evaluate the effects of shrinkage and creep on the truss, in particular on the initial prestressing state.

To measure strains and, consequently, prestressing forces produced in the elements by using Long's original prestressing technique, tests were performed using wedges. In addition, tests were done by traversing one span with a truck of known weight, with all the wedges loose and then with all wedges re-driven into place. This created data on the actual behavior of the bridge and a comparison could be made between the bridge behavior in prestressed and un-prestressed conditions.

Finite element analyses of the Eldean Bridge, using several linear elastic, plane frame models, were performed to study the static behavior of the bridge. The bridge was studied under the action of dead load alone; prestressing; moving, concentrated live loads; and a uniformly distributed live load.

Principal Observations

The experimental studies showed that the magnitude of the compressive forces produced in the counterbraces by prestressing them is about 5 - 6 kips. The additional stiffness gained by prestressing the counters, measured in terms of the reduction of mid-span displacement under live load, is equal to about 12 percent. Experimental influence lines for certain element forces and vertical displacements were also plotted, providing a basis for interpreting the actual behavior of the bridge. For example, the influence lines for vertical displacements at mid-span indicated some viscous effects in the bridge response under live load.

The portion of this study involving the viscous behavior of wood revealed a scarcity of generally applicable data. In practice, simple displacement amplification is often used to take these temporal effects into account. For the analysis of prestressed systems, there is a need for improved viscous models of wood behavior.

The numerical studies of the static behavior of Eldean Bridge showed that the maximum axial stresses under dead load were about 300 psi. Also, stresses from bending moments were less than one-tenth of those from axial forces, and shear forces in the members were almost zero. The maximum vertical displacements of the Eldean Bridge under dead load were equal to 0.64 inches without the action of the counterbraces, and 0.54 inches with it. The additional stiffness obtained with the counters is clear.

Regarding the prestress, the action of tightening a single wedge primarily affects only one panel. Prestressing does not cause an upward displacement of the truss; therefore, it does not relieve any of the dead load forces from the falsework. However, unless dead-load displacements are excessively large, a truss should be prestressed with the falsework removed and the dead load completely borne by the truss.

Influence lines of some main diagonals and counterbraces for a moving live load are shown. They were used to evaluate the minimum concentrated live load that would cause slackness in the diagonal elements of the truss. The live load required to loosen the central main diagonals with *inactive* counters was equal to 5.74 tons. A live load of about 14 tons was needed to loosen counterbraces of the panels adjacent to the end ones with *active* counters (note that the Eldean Bridge has no counters in the end panels). The same two elements were loosened, in the two bridge conditions, for uniformly distributed live loads of 38 lb/ft^2 and 46 lb/ft^2 on the half span of the bridge. These critical live loads were compared with design live loads used at the end of the nineteenth century.

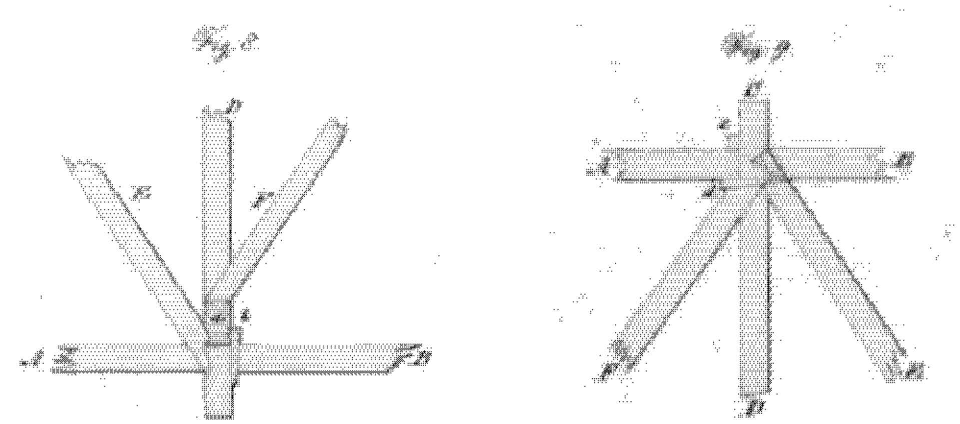
Longitudinal wood shrinkage, if uniformly distributed throughout the truss elements, does not cause significant stresses in the elements nor a loss of prestress in the counterbraces. It does cause downward displacements, whose magnitudes are very small. More detailed models are necessary to predict the effects of tangential and radial shrinkage at nodes.

Conversely, creep in the wood causes a loss of prestress and slackness in the counterbraces. This occurs because creep strains are not equal, since the members' dead- and live-load stresses are different. The first critical counters to lose prestress and become slack are those closest to the span ends. However, to predict the time-to-slackness, a viscous stress-strain model for wood that takes the changes in stresses over time into account would be needed.

Finally, numerical results and experimental measurements were compared, which indicated that linear, elastic, plane-frame models are adequate to predict the short-term behavior of wooden trusses, particularly the Eldean Bridge's Long trusses. Linear, elastic models with prescribed shrinkage and/or creep strains can only provide a general idea where and if slackness will occur, but they cannot give detailed information on time-to-slackness. The analyses did indicate that a Long truss needs periodic re-tightening of its wedges, and at a greater frequency during the early stages of its life, for it to continue to perform as designed.

Prestressing Techniques and Sequences

In Long's original design, braces and counterbraces were set into the truss, and held there by forces of compression. This happened by means of wedges (keys) located at different points of the truss panels. Keys were inserted behind the posts, opposite the ends of the diagonals, in order to force them into closer contact (element *bc* in Fig. 1a and element *c* in Fig. 1b). Other wedges were applied at the heads of the counterbraces. In the first patent of 1830, only a supporting block is shown at the lower end of the counters (element *a* in Fig. 1a), while the wedges that transfer the compressive forces to these counterbraces are located at their upper ends, between the counters and the center piece of the upper chord (element *d* in Fig. 1b). Long described the wedges as having a "thickness of 3/4 inch at the point; 1 1/4 inches at the butt, to a length of one foot."[19]



a) **b)**

Fig. 1: Connection of panel elements at the lower chord (a) and at the upper chord (b)[20]

After the construction of the main frame, "the bridge being raised and the scaffolding still remaining," the first keys to be driven were those at the ends of the post, used to force the main diagonals into closer contact to the posts.[21] The workmen drove these wedges, starting from the posts closest to center, and going toward the extremities of the span,

[19] Stephen H. Long, *Description of Col. Long's Bridges, Together with a Series of Directions of Bridge Builders* (Concord, NH: 1836).
[20] Ibid.
[21] Ibid.

beginning from the keys at the bottom of the posts, and subsequently driving the keys at the top of the posts.[22]

The last wedges to be inserted were the counterbrace wedges. Long wrote, "let the workmen begin at the extremities of the span, and proceed towards the centre, taking care to drive them as hard as they may be driven, with an ax or sledge weighing 4 or 5 pounds."[23] The prestressing technique Long specified for the counterbraces was conceptually simple, even if, in practice, it did not allow much control over the amount of prestress applied. Subsequent prestressing techniques using threaded iron rods would yield greater control with less physical effort.

As already noted, precompression of the diagonals induced corresponding states of pretension in the posts and chords. Without this precompression, the addition of live loads to the bridge would tend to put compression forces on the braces and tension forces on the counterbraces. Having no ability to handle tension loads, the joints at the ends of the counterbraces would tend to open up and become useless. With precompression loads greater in magnitude than any expected tensile loads, the compressive loads at these joints would be reduced, but not eliminated, and the joints would remain tight. Tight joints allowed all diagonals to contribute to the vertical stiffness of the bridge. Long was also aware that a certain percentage of prestressing would be lost due to timber shrinkage over time. Prestressing not only improved the structural performance of the truss, but it also counteracted a well-known property of wood.

A variant to the prestressing sequence described above was to build trusses on falsework with an upward bow, or camber. Removing the falsework after completion allowed the dead load to prestress the bridge as it tried to sag against the camber.[24] Long described this simple and effective procedure in his 1836 booklet, not as an actual prestressing technique, but rather as an explanatory demonstration of the "paradoxical" principle applied to his bridges (see Fig. 2), "that the truss frames … are subjected to no greater strain by the heaviest load admissible upon the bridge, than that to which they are constantly subjected, when the bridge is destitute of any load."[25]

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[22] Ibid.
[23] Ibid.
[24] See *American Wooden Bridges* (New York, NY: ASCE Historical Publication, 1976); F.E. Griggs and A.J. DeLuzio, "Stephen H. Long and Squire Whipple: The First American Structural Engineers" *ASCE Journal of Structural Engineering* 121, no. 9 (September 1995): pp.1352-1361.
[25] Long, *Description.*

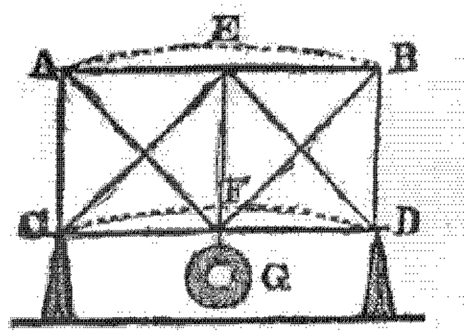

Fig. 2: Long's diagram to demonstrate the effect of prestressing

Details, Connections and Construction Technique for Long Trusses

The connections between timbers in Long's bridges are as crucial as the overall truss form and the prestressing sequence. To a great degree, they "affect the constructability, economy, performance, durability, safety, and beauty of a design."[26]

Compared to Town's lattice truss, Long's 1830 truss represented a significant step toward simplicity, not only in its concentration of functional elements, but also in its ease of construction.[27] Keyed compression joints and a limited number of bolted connections replaced the numerous treenailed connections in a Town truss.

The chords usually consisted of three or four parallel pieces, in order to allow an easy and efficient adjustment of the counterbraces. In his patent, Long detailed wooden pieces for chord splicing, but thinking ahead, he also proposed cast-iron splice blocks (Fig. 3). In both cases, these were pieces with lugs to engage notches in the different chord pieces. Wrought-iron bolts clamped the entire assembly together, and the joints were staggered. Barring shrinkage that could loosen the joint, the splice plates (not the bolts) carried the axial tension loads in the chords, but prestressing helped keep these joints tight as well. The splices between the pieces of the chords, which act as continuous timbers, have to be located in about the middle of the panel and have to occur *"... as remote as practicable from the points of greatest tension in the strings [chords], which are situated at the centre of the span, for the lower strings, and immediately above the piers for the upper strings."*[28] Long's chord-splicing detail was essentially that suggested by Navier; however, Long understood the critical importance of this detail and chose to devote much of his 1836 booklet to describing the chords and splicing details.[29]

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[26] Dario Gasparini and D. Simmons, "American Truss Bridge Connections in the 19th Century. I: 1829-1850," *ASCE Journal of Performance of Constructed Facilities* (August 1976), p.119.
[27] *American Wooden Bridges.*
[28] Long, *Description.*
[29] Gasparini and Simmons, "I:1829-1850," p.121

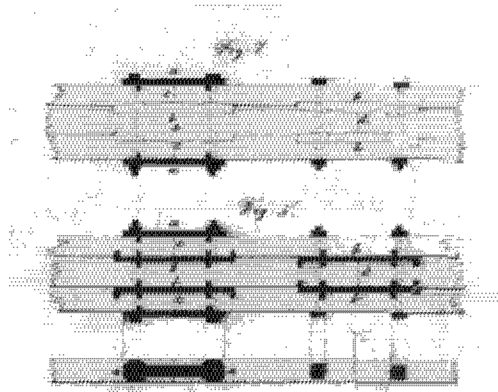
Fig. 3: Wooden and cast iron splicing of the chords[30]

Long's patent suggested that the best section for the posts was a square. He also indicated that each post should continue above the upper chord and below the lower chord for at least one foot. By cutting interlocking notches in posts and chords, they could be connected without the use of bolts (see Fig. 4). Long strongly recommended not bolting the chords to the posts.

The main diagonals were joined to the posts at their lower end with simple, bird-mouth connections. At the top ends, they fit into the same notches prepared in the chord to receive the posts. The counterbraces had tongues at their extremities. At their lower ends they were fixed into a block rising from the lower chord, or directly against the chord. On top, they terminated just short of the upper chord, leaving space for a wedge.

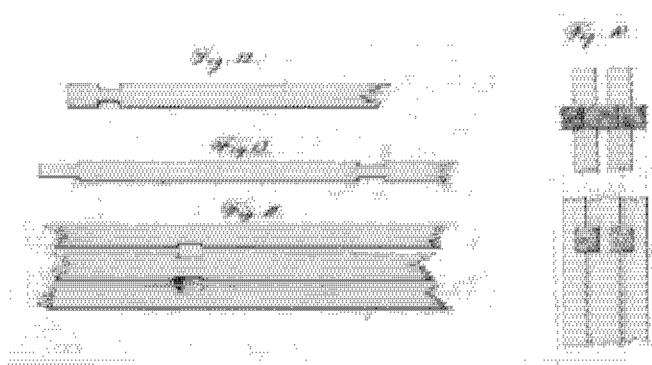
Fig. 4: Chord and post notches[31]

Long recommended dimensions for these notches and connections in his 1836 booklet, wherein he also described his various patented metallic details, including special shoulders and caps for the joints between braces and posts. These iron connections allowed him to avoid troublesome mortise-and-tenon joints with their susceptibility to rot and shrinkage.

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[30] Long, *Description*.
[31] Ibid.

Some of these metallic parts were incorporated into lateral bracing as well. In arranging the braces for lateral stability of the bridge, Long clearly stated, "shoulders, or steps, should invariably be regarded as the surest means …" and "no dependence should be placed in tenons, or mortices…."[32]

The function of the "superior arch braces" that he proposed in his 1830 patent was to assist the most-heavily loaded portion of the upper chords by carrying a portion of the compressive force. They were not, however, recommended for each bridge. In his 1836 booklet, Long gave directions, not only for the application of the prestressing load and the details of connections, but also for the general construction of the whole truss. Pre-assembly of trusses on land before erection on the falsework was common practice at that time. David Stevenson, a Scottish civil engineer making an extensive tour of the Eastern United States and Canada in 1837, noticed that, "the timbers of which Town's and Long's bridges are composed, are fitted together on the ground previous to their erection on the piers. They are again taken asunder, and each beam is put separately in the place which is to occupy, by means of scaffolding or centering timber."[33]

Present Condition of the Bridge

An on-site inspection of the Eldean Bridge during June 12-13, 2002, found almost 75 percent of the wedges to be loose. Some of these were a different species and dimensionally different from the typical wedges, indicating that they were likely replacements. According to an oral history, the slackness of counterbraces and the absence of several wedges were noted during repairs made in 1976. Additionally, about one-quarter of the post shoulders were found to be missing, broken or split. Both are signs that the periodic retightening recommended by Long has not always occurred.

Static Behavior of the Eldean Bridge

Issues Surrounding the Analysis of the Eldean Bridge

The preceding sections indicate the main issues surrounding an analysis of the Eldean Bridge. First of all, the initial states of stress obtained by driving the wedges on the counterbraces have to be clarified. The actual effectiveness of the prestressing technique with time has to be verified. When modeling the structure, therefore, the prestressing and its effectiveness (presence of active stiffening counterbraces) or ineffectiveness (inactive counterbraces, with the bridge behaving as if they did not exist) is the basic distinction to be taken into account. A short-term analysis of the bridge can only be carried out bearing in mind that, due to the high temporal dependency of the stress-strain behavior of wood, there also will be long-term effects caused by shrinkage and creep. If an analysis of shrinkage and creep effects is undertaken, some simplifying hypotheses can be accepted.

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[32] Ibid.
[33] *American Wooden Bridges.*

In two-dimensional models, only the effects of longitudinal shrinkage can be examined. Reasonably accurate strains due to creep can be calculated by assuming that the stresses in the truss members are constant.

Finite Element Analysis of Eldean Bridge

Computer analysis of two-dimensional, linear, elastic-frame models using a commercially available structural analysis program predicted the Eldean Bridge's structural behavior.[34] A two dimensional analysis was performed, based on experience to indicate that this would be sufficient to reveal the structural behavior. A frame model was used because of the continuity of the chords.

In the models, discrete elements joined at nodes represented the structure. In the case of a two-dimensional frame, each node has three independent components of displacement, or degrees of freedom (two mutually perpendicular displacements and one rotation).

Two models of the Eldean Bridge were used as shown in figures 5 and 6. Model A (Fig. 5) assumes the counterbraces are loose and inactive. This model reflects the actual condition of the bridge, as verified during the on-site inspection. Model B (Fig. 6) assumed that the prestressing was sufficient to allow the counters to carry both in tension and compression forces, as long as they remained under net compressive conditions. As in the real bridge, counterbraces were omitted from the end panels. The frame models were symmetric, so, as expected, symmetric loading conditions caused symmetric responses of the models.

Because of the similar nature of the two spans, an analysis of one truss would adequately represent the other three as well. The panel points were arbitrarily numbered from west to east, and these two figures represent the north truss of the east span. The member naming scheme derives from the numbers of the start and end nodes of each element. The final elements of the upper chord (U0U1, U11U12) and the final posts (L0U0, L12U12) were omitted from both models A and B because they do not participate in the load-carrying behavior of the truss. Their only function is to support the end portions of the roof.

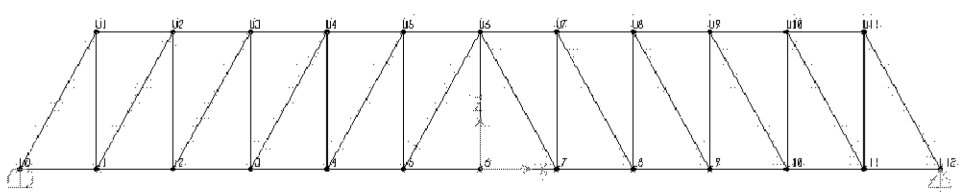

Fig. 5: Model A – Counterbraces inactive (loose)

[34] SAP 2000 NonLinear Version 6.11 for Windows, from Computer and structures, Inc., Berkeley, California

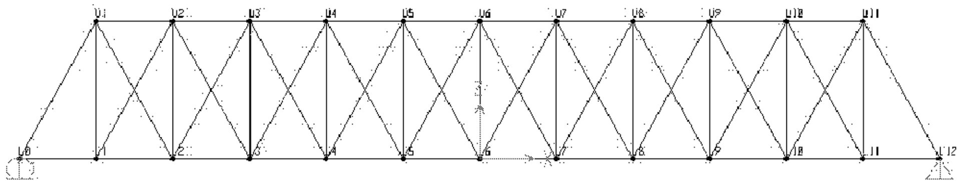

Fig. 6: Model B – Counterbraces active (sufficiently prestressed by wedges)

The primary difference between these two models is that Model A represents a statically determinate structure where forces and stresses can be calculated from principles of equilibrium alone. In Model B, the presence of the counterbraces makes the structure statically indeterminate, therefore equations of compatibility also must be satisfied to determine member forces. While in the first case an exact, manual analysis of a truss is practical, the computational effort required in the second case virtually demands the use of a computer program. Not having such resources, Navier and Long were forced to rely on the approximate solution methods available at the time.[35]

Dimensions and member cross sections were obtained from the 2002 HAER drawings of the bridge and by direct measurements taken on-site. Section properties calculated for each member are summarized in Table 1. They were also used for the estimation of dead loads, by assuming a unit weight for the wood.

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[35] C.L.M.H. Navier, *Résumé des leçons données à l'École des ponts et chausses, sur l'application de la mécanique à l'ètablissement des constructions et des machines* (Paris: Carilian-Goeury, 1833), and Long, *Description.*

Element	Type	Length (in)	Area (in^2)	Moment of inertia in plane (in^4)	Section modulus in plane (in^3)
L0L1, L1L2, L2L3, L3L4, L4L5, L5L6	lower chord	107	220	2218	403
U1U2, U2U3, U3U4, U4U5, U5U6	upper chord	107	199.5	1500	315
L2U1, L3U2, L4U3, L5U4, L6U5	counter	218	35	72	29
L0U1	diagonal	218	108	729	162
L1U2	diagonal	218	102	614	144
L2U3, L3U4	diagonal	218	90	421	112
L4U5, L5U6	diagonal	218	72	216	72
L1U1	post	190	120	1000	200
L2U2	post	190	104.5	785	165
L3U3, L6U6	post	190	114	857	180
L4U4	post	190	108	729	162
L5U5	post	190	102	614	144

Table 1: Section properties of the truss elements

A site inspection of the bridge revealed that I-beams (three for each bottom chord) now support the spans. At the abutments these I-beams are fixed in concrete and tied to the bottom chords (see Fig. 7). This arrangement can resist both vertical and horizontal displacements, so they were modeled as pin connections. At the central pier, each truss is similarly supported on I-beams, except that these connections allow for some degree of horizontal displacement. These were modeled as roller-type connections.

Fig. 7: Detail of the support at the abutment. Field photograph, 2002.

For the truss analysis, the following properties were assumed for white pine:

Modulus of elasticity, E = 1.2×10^6 lb/in^2
Poisson's ratio ν= 0.33

Both models A and B were used for analyzing the following loading cases:

1. dead load of the bridge
2. live load applied at different panel points
3. prestressing loads obtained by driving the wedges against the counterbraces
4. dead load with prestressing and live loads applied at different panel points
5. dead load with prestressing and effects from shrinkage and creep in the wooden members

Analysis of the Eldean Bridge Under Dead Load

The structure's dead load was calculated from the member properties, assuming unit weights of 24.5 lb/ft^3 for white pine, 44 lb/ft^3 for red oak, and 34.9 lb/ft^3 for American elm. The analytical program automatically computed the weights of the truss members; however, a multiplier of 1.1 was applied to the truss weight to compensate for the miscellaneous wooden and metal elements. One-half of the deck and the roof weight were distributed to each truss. The dead load of the roof (1.15 kip) and of the deck and the siding (2.2 kip) were applied at the panel points. A summary of the total dead load of one truss of the Eldean Bridge is listed in Table 2.

WEIGHT OF THE TRUSS					
Element:	upper chord	lower chord	post	diagonal	counter
area (in^2)	199.5	220	113.4	93	35
length (in)	107	107	190	218.1	218.1
weight (lb)	298.9	329.6	301.7	283.9	106.8
quantity/bay	1	1	1	1	1
weight/bay (lb)	298.9	329.6	301.7	283.9	106.8
Weight per bay of the truss (lb): (self weight)				**1321**	

WEIGHT OF THE ROOF								
Element:	bearing beam	rafters	planks	Post	crossing brace	diagonal brace	decking	metal sheet
area (in^2)	22.75	14	6	6	48	22.5	80.25	144*
length (in)	107	190	107	73	208	233.9	190	
weight (lb)	34.1	37.2	9.0	6.1	139.8	73.7	213.5	2.5**
quantity/bay	1	5	17	2	1	1	1	
weight/bay (lb)	34.1	186.2	152.8	12.3	69.9	73.7	213.5	360
Weight per bay of the roof (lb): (concentrated load on upper panel points; *: ft^2, **: lb/ft^2)						**1102**		

WEIGHT OF THE DECK					
Element:	bearing beam	floor beam	diagonal planks	crossing planks	diagonal brace
area (in^2)	102	27	160.5	240.75	27.5
length (in)	254	107	104	104	234
weight (lb)	710.7	75.1	434.0	651.0	167.3
quantity/bay	0.5	4	1	1	1
weight/bay (lb)	336.8	300.5	333.8	500.8	128.7
Weight per bay of the deck (lb): (concentrated load on lower panel points)				**1601**	

WEIGHT OF THE SIDING				
Element:	Planks	hor. exterior	hor. int	parapet
area (in^2)	160.5	8.75	8.75	33.75
length (in)	176	107	107	107
weight (lb)	395.5	13.1	13.1	50.6
quantity/bay	1	3	3	1
weight/bay (lb)	395.5	39.3	39.3	50.6
Weight per bay of the siding (lb) (concentrated load on lower panel points)			**525**	
Total weight of a bridge span:			**109.2 (kips)**	

Table 2: Weight (dead load) of the Eldean Bridge

Fig. 8 shows the axial forces occurring in models A and B when subjected to their corresponding dead loads. The sign convention selected indicates forces producing tension in members as positive and forces producing compression in members as negative.

In both models, the chords have significant axial forces—compression in the upper chord and tension in the lower chord—that increase from the ends to the center of the span. For both models and for both chords, the absolute value of the maximum axial forces is about 46 kips. The vertical and diagonal elements have increasing axial forces from the mid-span to the ends of the bridge. Bridge constructors seem to have understood this general behavior, as they often reduced the sectional areas of diagonals and posts toward the center of the bridge spans.

For all elements, the maximum absolute values of bending moments and shear forces are reached in members close to the bridge ends (see respectively Fig. 9 and Fig. 10). A summary of the forces in the main elements of the truss models A and B is shown in Table 3.

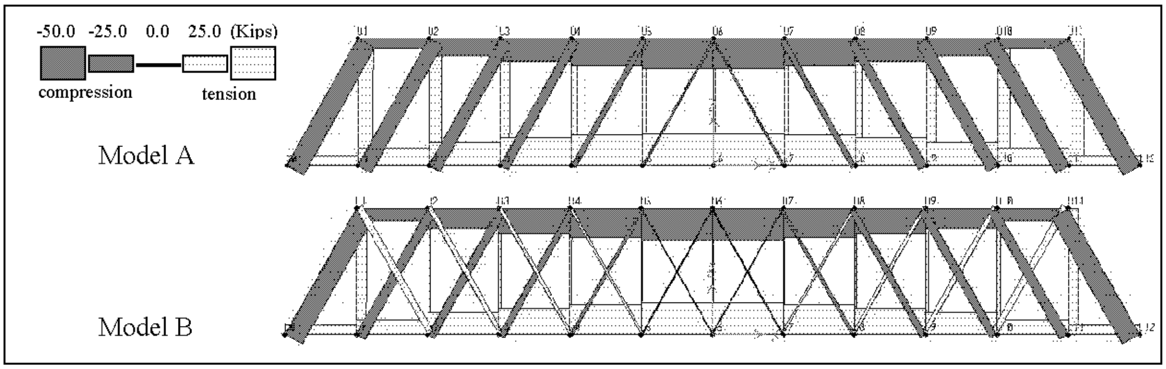

Fig. 8: Axial forces under dead load in the frame members of the two models

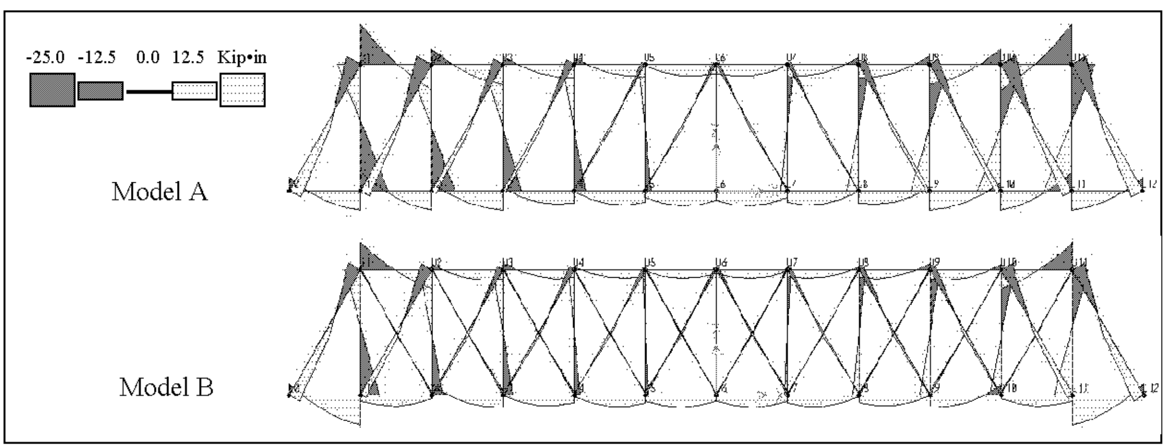

Fig. 9: Bending moments under dead load in the frame members of the two models

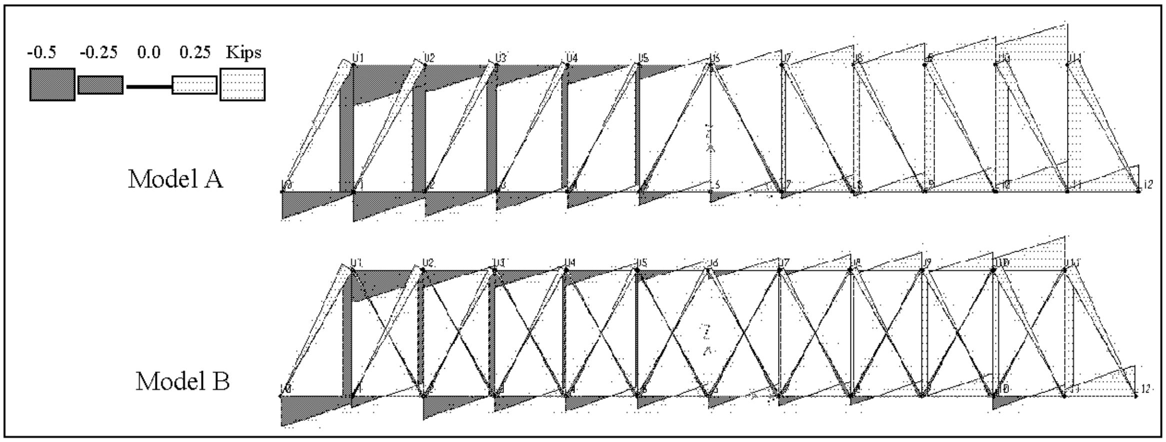

Fig. 10: Shear forces under dead load in the frame members of the two models

Table 3, Below: Summary of the forces due to dead load in the main elements of the truss models A and B

	Element / Location		L5L6 chord central panel	U5U6 chord central panel	L1L2 chord final panel	U1U2 chord final panel	L0L1 chord final panel	L1U1 post final panel	L1U2 diagonal final panel	L2U1 counter final panel	L0U1 diagonal end truss
Model A	max axial force	kip	46.80	-45.54	25.97	-14.52	14.32	23.27	-23.31	--	-29.16
Model A	max positive bending mom.	kip• in	11.40	8.99	14.87	19.14	14.39	17.42	7.62	--	9.83
Model A	max negative bending mom.	kip• in	--	--	-14.06	-29.77	-9.83	-20.83	-12.86	--	-12.35
Model A	max shear force	kip	-0.20	-0.22	-0.45	-0.63	-0.41	-0.20	0.18	--	0.19
Model B	max axial force	kip	47.44	-47.24	21.93	-19.48	14.60	15.43	-14.71	9.72	-29.75
Model B	max positive bending mom.	kip• in	9.22	6.62	8.27	14.35	22.33	11.44	4.15	0.55	8.43
Model B	max negative bending mom.	kip• in	--	--	--	-20.67	-8.43	-14.04	-8.96	-1.32	-9.96
Model B	max shear force	kip	-0.19	-0.18	0.20	-0.49	-0.47	-0.13	0.14	-0.03	0.17

Regarding forces, shear is almost null and its highest value is about twenty times smaller than the highest axial force. Stresses due to bending moments are an order of magnitude smaller than stresses due to axial forces. Therefore, despite the continuity of the chords, a truss model with perfect-pin connections may be sufficient to study the bridge behavior.

Regarding the differences between the two models, it can be said that under dead load, the magnitude of the axial forces in the horizontal members does not change significantly (about 2 percent). The counterbraces allow a redistribution of the forces in the diagonal and vertical elements, however, causing a reduction in axial force of about 30 percent in the last panel diagonal and post.

The stiffening effect of the counterbraces on the truss's behavior is clearly demonstrated by the lower nodal displacements. The vertical displacement of the mid-span node on the lower chord, which is the highest in the case of symmetric loading like this dead load, decreases from 0.64 inches in Model A to 0.54 inches in Model B. These values of vertical displacement will increase in time due to creep.

Analysis of the Eldean Bridge Under Live Loads

A live load analysis was performed by applying a concentrated unit gravity load (1 kip) on each node along the length of the truss to both models A and B. A live load analysis was also performed by applying a set of unit live loads over one-half the span as well as the entire span, corresponding to a uniformly distributed load of 12.94 lb/ft^2.

The results of the analyses were scaled to reflect nineteenth century design live loads. Ketchum described design live loads for bridges at the end of the nineteenth century.[36] His own specifications were the same as those used by both the American Bridge Company and Theodore Cooper. For "ordinary country highway bridges," Ketchum prescribed for the trusses, "a load of 80 lb/ft^2 of total floor surface for spans up to 75 feet; and 55 lb/ft^2 for spans of 200 ft and over; proportionately for intermediate spans." Ketchum also described the specifications of J.A.L. Waddell and reproduced Waddell's graph for live loads for different classes of bridges. Class C applied to bridges in "light country service." For a 100' span Waddell prescribed a uniform load of 70 lb/ft^2 but also required "a concentrated load of 10,000 lbs equally distributed upon two pairs of wheels, the axles of which are 8 ft. apart, and the central planes of the wheels 6 ft. apart."[37]

[36] M.S. Ketchum, *The Design of Highway Bridges and the Calculation of Stresses in Bridge Trusses* (New York: The Engineering News Publishing Co., 1909).
[37] J.A.L. Waddell, *The Designing of Ordinary Iron Highway Bridges* (New York, NY: J.Wiley, 1884).

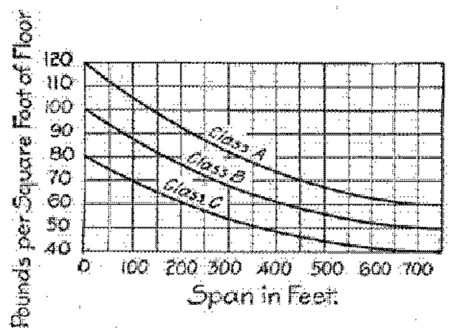

Fig. 11: Waddell's live loads for highway bridges[38]

Earlier in the nineteenth century, before the existence of model specifications, engineers and builders needed to determine reasonable live loads for their bridge designs. For example, Charles Ellet, in his "Report on the Wheeling and Belmont suspension bridge" (1847), writes, "it is not easy to imagine that a greater load can ever be brought on the flooring of this bridge than that which would be occasioned by covering the carriage-way with as many teams as could stand upon it. A column of sixteen of the six-horse wagons used on the National Road, would occupy the bridge from one abutment to the other."[39] He evaluated the weight per linear foot due to a double line of wagons, fully loaded, and added the weight of 500 persons, obtaining the "greatest transitory load which need be provided against" of 618 lb/ft.[40] He also noted that the total weight of such a load on his 960-ft. long bridge would be equal to the weight of 700 head of cattle, or to that of an army of 4,000 men. By dividing the weight per linear foot by the width of the Wheeling Bridge (about 19'), it is found that he was designing his bridge for a uniformly distributed live load of about 32.5 lb/ft^2. Ellet's live load values have since been used for structural analyses of several historic bridges.[41]

A presumption of linear-elastic behavior by the bridge allows the superposition of various live and dead load conditions. In this case the results of the live load analysis can be scaled for other values of live load, since this basic behavior does not change. The live load axial forces in the truss members are plotted in figures 12 and 13 with the live load applied at panel points L2, L4, and L6. Fig. 14 shows member forces for a uniform load applied on one-half of the span. Note that the scales of these figures vary to achieve reasonable proportions. The member forces produced by applying only unit live loads are, in fact, an order of magnitude smaller than those produced by the dead load alone.

[38] Ketchum, *Design of highway bridges.*
[39] C. Ellet, Jr., *Report on the Wheeling and Belmont Suspension Bridge, to the City Council of Wheeling* (Philadelphia: John C. Clark Printer, 1847).
[40] Ellet.
[41] E.L. Kemp and J. Hall, "Case Study of Burr Truss Covered Bridge," *ASCE Engineering Issues—Journal of Professional Activities* 101, no. E13 (July 1975): pp. 391-412.

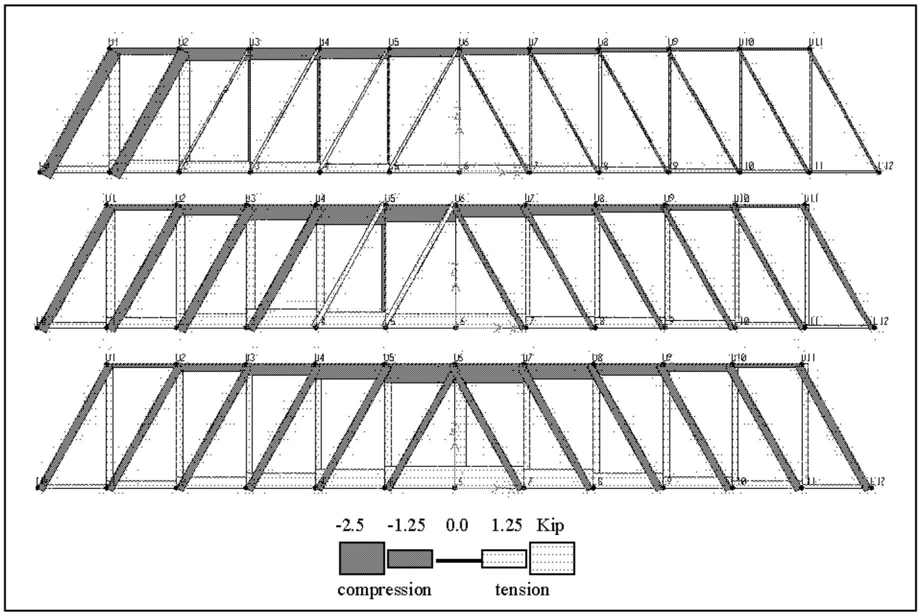

Fig. 12: Model A: axial forces under live loads applied in L2, L4 and at mid span

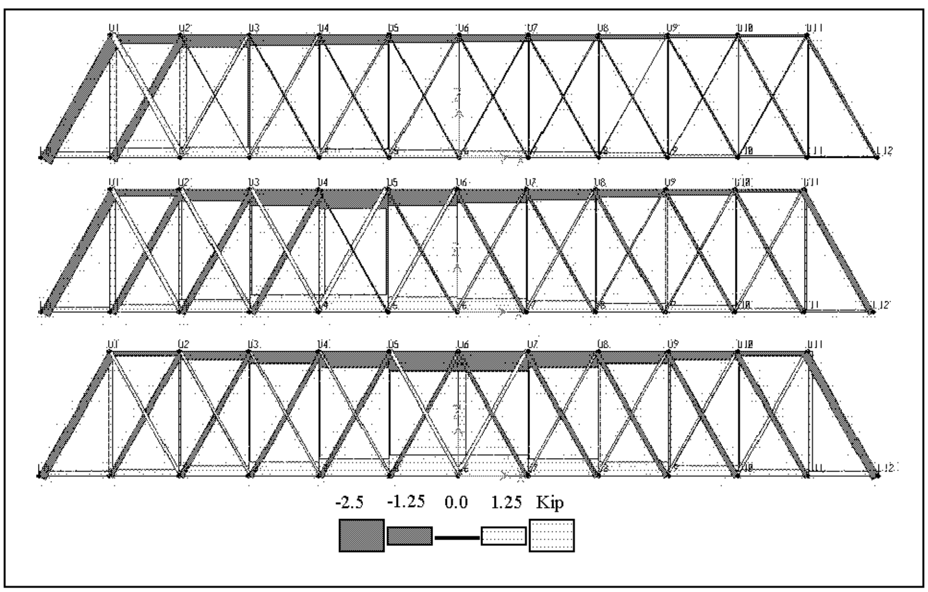

Fig. 13: Model B: axial forces under live loads applied in L2, L4 and at mid span

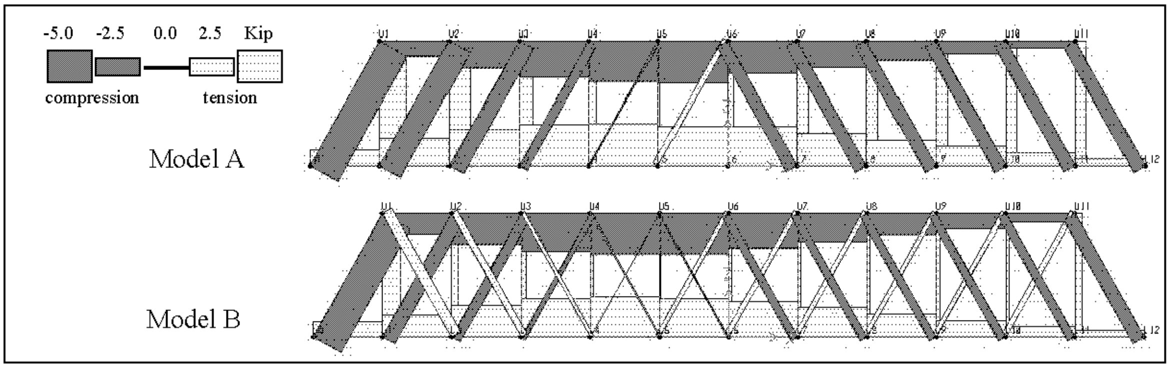

Fig. 14: Model A and Model B: forces under a uniformly distributed live load over one half of span

In the analysis with uniformly distributed load on the entire span, the central elements of the upper chords are the most heavily loaded axially. An axial force of -9.84 kips and a bending moment of 2.45 kip·in are found in elements U5U6 and U6U7. Scaling these forces to add a reasonable value of uniformly distributed live load (40 lb/ft^2 was chosen), the central elements are subject to a compressive stress of -177 psi. The corresponding value of compressive stress due to dead load is -257 psi. Axial force and bending moment in the central elements of the upper chord are reported in Table 3, while section areas and moduli of the elements are listed in Table 1. Thus, for reasonable values of live load, the stresses due to dead and live loads are of the same order of magnitude. The total compressive stress found, -434 psi, is considerably lower than the maximum allowable stress found in the National Design Specification for Wood Construction. Fig. 13 shows that the live load at L4 produces tensile forces in both main diagonals and counterbraces. A uniformly distributed load over a half-span also produces tensile forces in main diagonals and counterbraces, as can be seen in Fig. 14. To prevent diagonals from becoming loose, the combined action of dead load and prestressing must produce compressive forces that exceed the tensile forces produced by design live loads.

The behavior of the diagonal members for a live load applied at any point on the bridge is also shown through influence lines, which plot the response-force of a particular member versus the location of the live load. The net axial forces due to live load application were considered. Influence lines for a unit live load are plotted for elements L1U2, L3U4 and L5U6 (see Fig.), and for counters L2U1, L4U3 and L6U5 (Fig.).

Fig. 15 shows that a live load can cause both tensile and compressive forces in the main diagonals. Forces in the diagonal elements of Model B are lower than those of Model A, because of the presence of the counters. Fig. 16 shows that a live load causes mainly tensile forces in the counterbraces.

Fig. 17 shows vertical displacements of three panel points on the lower chord (L2, L4, and mid-span point L6) for different live loads positions. When the live load is moving from the first panel point to a particular node, the node displacement increases more or less linearly. Correspondingly, when the load is moving away from the node, a nearly linear decrease of displacement occurs, except for node L2. Fig. 17 also clearly shows that the displacements obtained with counterbraces (Model B), are lower than those without counterbraces (Model A).

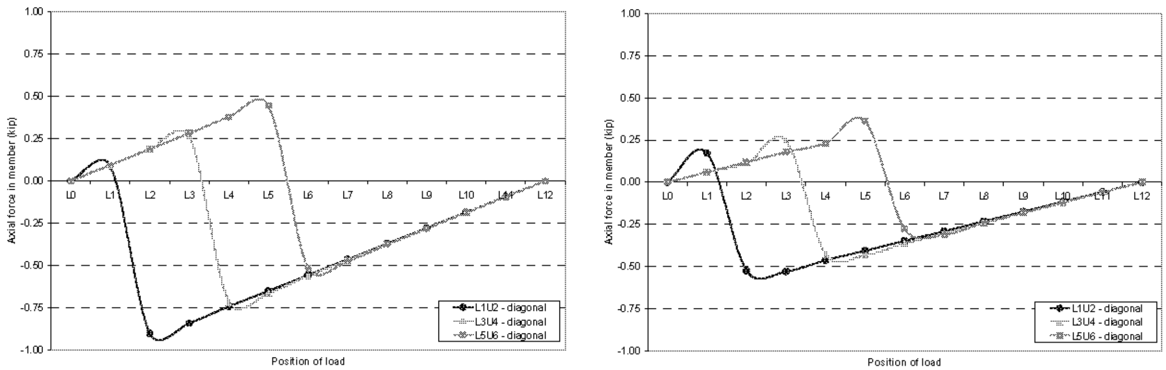

Fig. 15: Model A (left), Model B (right): influence lines of three diagonal members, showing the unit live load axial forces as a function of load position

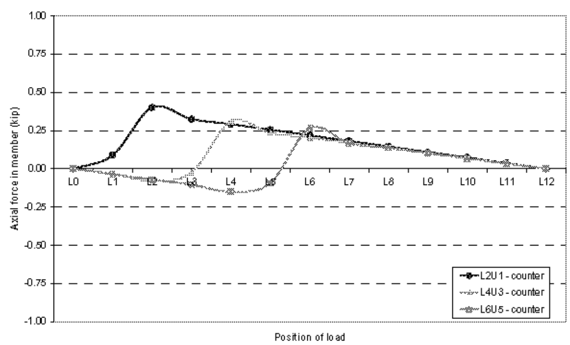

Fig. 16: Influence lines of three counters, showing the unit live load axial forces as a function of load position

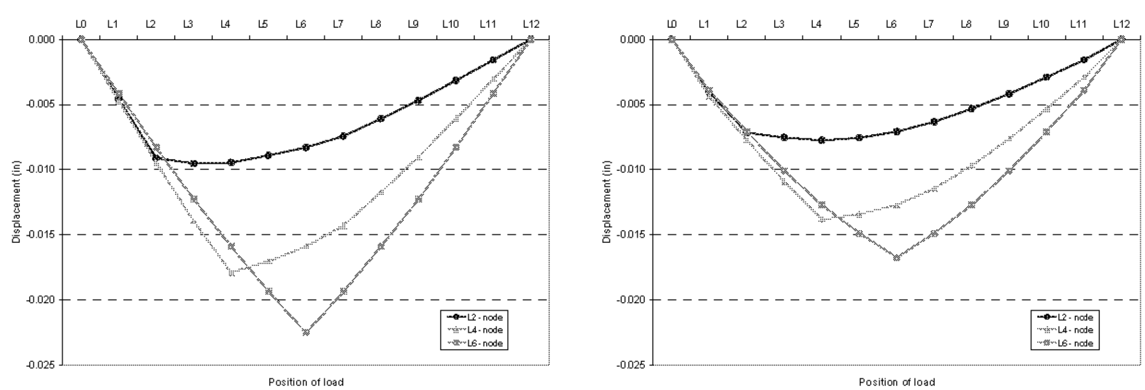

Fig. 17: Model A (left), Model B (right): influence lines of three nodes of the lower chord, showing the displacements under a unit live load as a function of load position

The influence lines for the main diagonal members show that the element subjected to the highest axial tension force for the action of a live load application is L5U6, the mid-span diagonal, when the load is applied at node L5. Since the central diagonals are the members subjected to the lowest axial compression due to dead load, they are the members at greatest risk of a zero net axial force due to combined dead and live loads at L5, with the corresponding loosening of diagonal L5U6. Due to the symmetry of the structure, this also applies for diagonal L7U6 when a live load is applied in L7.

Recall that the axial force due to dead load in diagonal L5U6 is -2.56 kip (compression), and that a live load of 1 kip at L5 produces an axial force in the member of 0.45 kip (tension). Proportionally, the live load equal to 5.74 kip at L5 (equivalent to a total of 5.74 tons for both trusses) will produce a tensile force of 2.56 kip in diagonal L5U6, resulting in a zero net axial force.

The uniformly distributed load that can loosen diagonal L5U6 may be similarly calculated. A set of six live loads of 1 kip applied at panel points L1 to L6 produce an axial force in L5U6 of 0.87 Kip (tension). The set of live loads capable of producing a tensile force of 2.56 kip in the diagonal member is equal to 2.94 kip applied to each panel point on half span. This is equivalent to a uniform live load of about 38 lb/ft^2 on one-half of the bridge.

Under such a concentrated (5.74 tons on the bridge) or uniformly distributed (38 lb/ft^2 on half span) live load, the structure would become a kinematic mechanism, assuming the hypothesis of perfect pin connections between the truss elements. In reality, the joints of a bridge are not perfect pins. For example, the lower and upper chords are continuous. Because of this continuity of the chords, the bridge does not actually become a mechanism under live loading, but it is clear that this value represents a “critical load” for the structural behavior of the bridge.

Note that the minimum uniformly distributed live load capable of causing slackness in the diagonals is lower than the design loads in late nineteenth century model specification, but slightly higher than that used by Ellet in 1847. Long introduced prestressing in compression to reduce the likelihood that live loads could completely unload, and thus loosen, any member.

Prestressing Analyses of the Eldean Bridge

In Long’s trusses, the counterbraces were prestressed by driving wooden wedges between them and the bottom chord. Model B analyzed the effects of prestressing on the truss. As above these analyses were performed by applying 1 kip live loads to the nodes of the structure. The prestressing action of the wedges was modeled by applying additional loads at the nodes. Different analyses were carried out by first considering each panel prestressed by itself, then all panels prestressed concurrently. Fig. 18 shows the internal (a) and external (b) loads on one typical counter, assuming all nodes to be fixed,

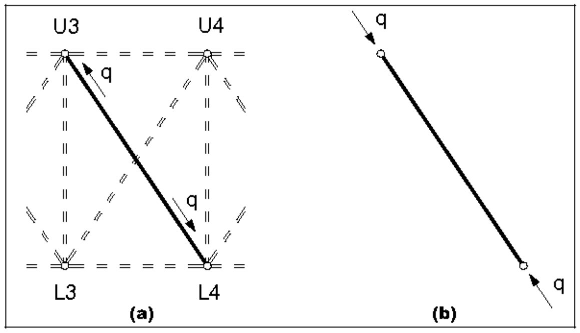

Fig. 18: Effective nodal loads from prestressing the counterbrace (a); corresponding fixed-end forces on the counterbrace (b)

To determine the actual axial force in a counter (n_i), the force in the element from the nodal displacements caused by the effective nodal loads (μ_i) must be superposed onto the fixed-end force (q) as follows:

$$n_i = \mu_i + q$$

Fig. 19 shows axial forces in a portion of the model centered on panel L3U3-L4U4, due to effective nodal load from prestressing counter L4U3. Counter L4U3 is subjected to an axial tensile force of 0.43 kip due to the effective nodal loads and to a fixed-end compressive force of -1 kip, for a net axial compressive force of -0.57 kip. The two diagonals, therefore, are precompressed with an axial force more or less equal to half of the nodal load applied (-0.57 kip the counter and –0.53 kip the main diagonal). Conversely, both the posts (0.43 kip) and the chords (0.27 kip) are in tension. As predicted, prestressing has the greatest effect on the diagonals. The forces on the posts are somewhat less, but still comparable in absolute value to those on the diagonals. The magnitude of forces on the chords is only about half of that on the diagonals. The distribution of axial forces in the different elements obtained by prestressing the counterbrace is a function of the panel's geometry.

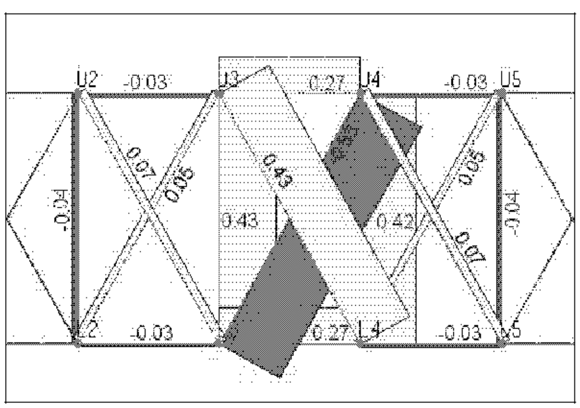

Fig. 19: Axial forces (kips) from effective nodal loads generated by prestressing the element L4U3. The net force in L4U3 can be found by superposing the -1 kip fixed-end force onto this 0.43 kip force.

The effect of prestressing a single counter rapidly decreases to zero in nearby panels (see Fig. 19). In the two adjacent panels, the magnitudes of axial forces are an order of magnitude smaller than in the prestressed panel, and they fall to almost zero in the subsequent panels. To achieve the advantages of prestressing in a whole truss, all of its panels have to be prestressed.

Fig. 20 shows the axial forces from effective nodal loads from prestressing all the counterbraces. The actual axial forces in the counterbraces can be calculated by superposing the fixed-end forces and the axial forces produced by the effective nodal loads.

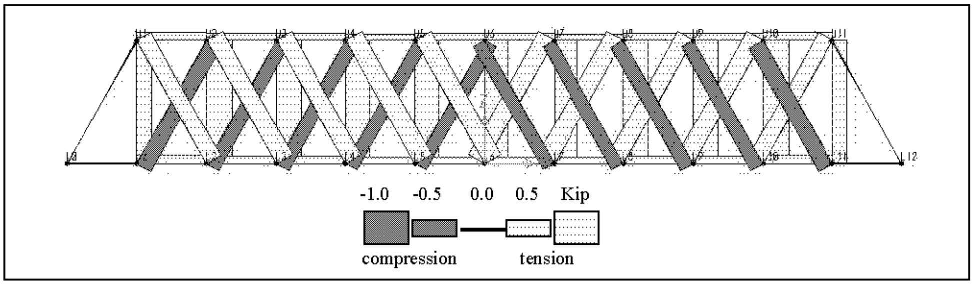

Fig. 20: Axial forces from effective nodal loads generated by prestressing all the counters. The net force in each counter can be found by superposing the -1 kip fixed-end force onto these forces.

The distribution of forces due to prestressing is more or less the same in all of the truss panels. Therefore, the effect of the prestressing with the simultaneous action of a live load should be analyzed in particular for the following truss elements:

- the lower chord element and post subject to the greatest tension, where the prestressing causes an increase of tension
- the main diagonal subject to the greatest compression, where the precompression of the counter produces an increase in compressive force.
- the most highly stressed counter, to see if the effect of precompression is enough to avoid a null net axial force with the consequent loosening of the counter.

The sequence of node displacements with the application of the prestress nodal loads is shown in Fig. 21. The prestressing sequence is that prescribed by Long, starting simultaneously from the extremities of the span and proceeding towards the center. The last point in each of the three curves represents the total displacement of nodes L2, L4, and L6 (mid-span) produced by prestressing the entire truss. Their displacements are 0.003 inch, 0.007 inch, and 0.012 inch, respectively.

The displacements are downward, but at least for the 1 kip effective load, they are very small relative to the predicted dead load displacements. Because the displacements are downward, if wedges are driven with falsework in place, the prestressing action does not have the effect of relieving some of the dead load forces on the falsework. Also, since the dead load produces tensile forces in the counters, it is sensible to prestress a truss after the falsework has been removed and the dead load is being carried by the truss alone, unless displacements resulting from the dead load are too large.

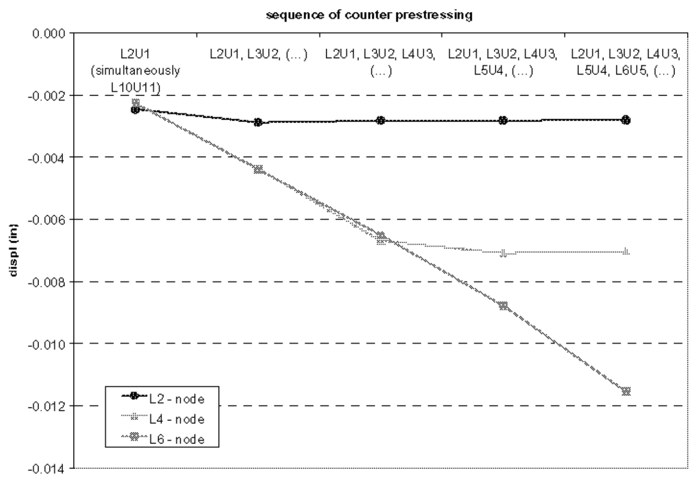

Fig. 21: Nodal displacement of the lower chord, with the progression of prestressing.

Analyses Under Dead and Live Loads, With the Effect of Prestressing

During the experimental studies the actual axial forces produced by driving in wedges at counterbraces L3U2 and L5U4 were recorded. They were respectively equal to -5.648 kip and -6.375 kip. These actual prestressing forces in the counters (N_i) were used to evaluate the actual prestressing nodal loads applied to the truss. Considering the linear relation between nodal loads and member forces and using the results already found in the case of unit prestress loading, the actual prestressing nodal loads (Q) were calculated as follows:

$$Q = \frac{N_i}{(\mu_i + q)}$$

The effective nodal loads applied to the truss by counters L3U2 and L5U4 are 9.38 kip and 11.840 kip, respectively. For the analysis of the whole truss, an average value of nodal prestressing loads equal to 10.889 kip was used. The results of the analyses under unit prestress loading and the evaluation of the actual nodal loads are tabulated below.

Counterbrace	Axial force due to unit nodal loads μ_i (kip)	Fixed end force q (kip)	Actual axial force n_i (kip)	Experimental axial force N_i (kip)	Effective nodal loads Q_i (kip)
L2U1	0.424	-1	-0.576	--	--
L3U2	0.432	-1	-0.568	-5.648	9.938
L4U3	0.430	-1	-0.570	--	--
L5U4	0.462	-1	-0.538	-6.375	11.840
L6U5	0.458	-1	-0.542	--	--

Table 4: Axial forces in the counterbraces due to a unit prestress load and effective nodal loads evaluated from the experimental axial forces

Counterbrace	Axial force due to effective prestress nodal loads (kip)	Fixed end force (kip)	Actual axial force (kip)
L2U1	5.327	-10.889	-5.562
L3U2	6.140	-10.889	-4.749
L4U3	5.978	-10.889	-4.911
L5U4	6.382	-10.889	-4.507
L6U5	6.124	-10.889	-4.765

Table 5: Axial forces in the counterbraces due to the effective prestress loading

Table 6 shows forces for the case of only dead load and for the case of simultaneous dead and prestressing loads. It can be seen that the prestress loading caused a decrease of the axial compressive force due to dead load in the upper chord and an increase of the axial tensile force in the lower chord. Both effects varied in intensity with position. In the end panels, the change of axial force amounted to almost 20 percent of the original force values, where in the central panels the average difference was only about 5 percent.

The prestress loading also caused increases in compression in the diagonals and tension in the posts. These increases were highest in the central panel, where the initial axial forces were lowest. Finally, the counterbraces were subjected to a compressive axial force varying from -4.78 kip (L6U5) to -5.57 kip (L2U1). The downward mid-span displacement decreased from 0.64 in. under dead load alone to 0.12 in. with prestressing, an 81-percent improvement.

Element		L5L6 Chord	U5U6 Chord	L6U6 Post	L5U6 Diag.	L6U5 Count.	L1L2 Chord	U1U2 Chord	L1U1 Post	L1U2 Diag.	L2U1 Count.
Location		Central panel					End panel				
Axial force under dead load	kip	46.80	-45.54	2.69	-2.56	--	25.97	-14.52	23.09	-23.16	--
Axial force under prestressing	kip	2.16	2.28	7.82	-4.26	-4.78	2.70	2.66	4.82	-5.31	-5.57
Axial force under dead load and prestressing	kip	48.96	-43.26	10.51	-6.82	-4.78	28.67	-11.86	27.91	-28.47	-5.57

Table 6: Axial forces under dead load and prestress load

An analysis of the truss under the simultaneous actions of dead, live, and prestressing loads was used to determine the magnitude of live load that could loosen the prestressed counters.

With no live load on the truss, the prestressed counters with the lowest value of compressive axial force are the central ones (L6U5, L6U7). They have an axial force equal to -4.78 kip under prestressing and a tensile axial force equal to 0.27 kip when a unit (1 kip) live load is applied at L6. Conversely, the counterbraces with the highest compressive axial force are L2U1 and L10U11 (-5.57 kip), but they also have the highest tensile axial force (0.40 kip) when a unit live load is applied to panel points L2 or L10, respectively. (Figure 16 shows the unit live load axial forces in the counterbraces as a function of load position.)

The end and central panel conditions represent the two extremes for the addition of a live load to unload and loosen a counter—the former for being the one most affected by the presence of a live load, and the latter for being the least affected by prestressing. Assuming linearity and using superposition, the minimum live loads capable of loosening these counterbraces can be calculated. A live load of 13.93 kip applied at truss node L2 can loosen counterbrace L2U1 (similarly, this load at node L10 can loosen counterbrace L10U11), and 17.98 kip applied at node L6 can loosen counterbraces L6U5 and L6U7. Numerical analysis confirmed these results.

The same procedure can be employed to calculate the effects of a uniformly distributed live load. A set of six unit live loads applied at panel points L1 to L6 produced a tensile axial force of 1.58 kip in counter L2U1. By proportion, it was determined that 3.53 kip applied to each panel point on the half span, equivalent to a uniform load of about 46 lb/ft^2 on half of the bridge, would produce a tensile force of 5.57 kip in counter L2U1, making its net axial force equal to zero.

The 13.93 kip minimum concentrated live load capable of loosening a counterbrace corresponds to a live load of about 14 on the bridge, which is approximately three times the current load limit. The minimum distributed live load required to loosen a counter is equivalent to 46 lb/ft^2, a value 42 percent higher than the design live load specified in Ellet's 1847 report.

While it is vital that the counters remain under compression, the magnitude of that compression is of little importance to the truss's performance. As long as the compression force in the counters from prestressing is not reduced to zero by a live load, the counters will remain effective in helping to carry the live load and stiffen the bridge.

Effects of Shrinkage and Creep on Bridge Behavior

Shrinkage and creep are deformations related, respectively, to changes in moisture content and the behavior of wood under load. Both shrinkage and creep are time-dependent phenomena. Shrinkage involves a reduction in dimensions of the truss members as the wood looses moisture and, thus, volume. It is a normal and typical process in the wooden elements used to build bridges, especially those built using wood that was still green.

In reality, if a wooden element is restrained, its shrinkage will be hindered and the restraints will induce a tensile stress on the element. Therefore, the corresponding behavior of the truss under shrinkage can be evaluate by computing the forces in the truss elements when all the shrinkage nodal loads are applied, and then superposing the effect of the fixed-end forces, as done in the prestress analyses.

In the case of creep, a temporal increase of the initial elastic strain due to load occurs. This strain will be negative for elements under compression and positive for those under tension. Thus, if creep is hindered, the effects will be the addition of compressive loads in tensioned elements and tensile loads in those under compression. Except for the direction of some forces, the same method used for shrinkage can be used to calculate the behavior of the truss under creep.

Both creep and longitudinal shrinkage change the physical dimensions of truss members, cause displacements, and, possibly decreases in prestressing forces in the counterbraces. Therefore, some analyses of the truss under the combined effects of shrinkage and creep were carried out, using Model B, to evaluate their influence on the initial prestress state. For the shrinkage analysis, a value of strain equal to 0.002 was used. This is equal to the longitudinal shrinkage, from green to oven dry, for a large number of wood species. This value could be reduced to take into account the fact that the actual shrinkage of the bridge elements is from a green moisture content of 30 percent to something in the range of 12 to 19 percent, depending on a variety of circumstances. However, considering that the tangential and radial shrinkage at the nodes was not modeled, the conservative value of 0.002 was used.

The effective nodal loads (S_i) due to longitudinal shrinkage were calculated for each element as follows, where ε_s is the shrinkage strain, E the modulus of elasticity of wood, and A_i the area of the *i*-element:

$$S_i = \varepsilon_s \cdot E \cdot A_i$$

The results of the shrinkage analyses are listed in Table 7. The actual axial forces from shrinkage are almost null in all the elements. This is because the entire structure is free to shrink and the shrinkage is uniformly distributed in all the elements. A shrinkage analysis performed on Model A also produced essentially no axial forces.

Element		Geometry		Dead load & prestressing		Shrinkage		
Name	Type	Length (in)	Area (in2)	Axial force prest. (kip)	Axial force dead & p. (kip)	Fixed end axial force (kip)	Axial force from nodal loading (kip)	Actual axial force (kip)
L0L1	chord	107	220	0.03	14.35	528.00	-528.001	-0.0008
L0U1	diagonal	218	108	-0.04	-29.05	259.20	-259.201	-0.0013
L1L2	chord	107	220	2.70	28.67	528.00	-527.999	0.0015
L1U1	post	190	120	4.82	27.91	288.00	-287.999	0.0013
L1U2	diagonal	218	102	-5.31	-28.47	244.80	-244.8	0.0004
L2L3	chord	107	220	2.30	37.39	528.00	-527.995	0.0053
L2U1	counter	218	35	-5.57	-5.57	84.00	-83.9995	0.00054
L2U2	post	190	104.5	8.74	27.28	250.80	-250.799	0.0015
L2U3	diagonal	218	90	-4.61	-22.71	216.00	-216.003	-0.0032
L3L4	chord	107	220	2.39	44.00	528.00	-527.99	0.0099
L3U2	counter	218	35	-4.76	-4.76	84.00	-83.9983	0.00172
L3U3	post	190	114	8.32	22.37	273.60	-273.599	0.0006
L3U4	diagonal	218	90	-4.76	-17.71	216.00	-216.004	-0.0035
L4L5	chord	107	220	2.22	47.71	528.00	-527.983	0.0167
L4U3	counter	218	35	-4.92	-4.92	84.00	-83.9985	0.00152
L4U4	post	190	108	8.10	17.60	259.20	-259.2	0.0003
L4U5	diagonal	218	72	-4.44	-12.12	172.80	-172.805	-0.0045
L5L6	chord	107	220	2.16	48.96	528.00	-527.976	0.0238
L5U4	counter	218	35	-4.52	-4.52	84.00	-83.9978	0.00218
L5U5	post	190	102	7.87	12.88	244.80	-244.8	-0.0003
L5U6	diagonal	218	72	-4.26	-6.82	172.80	-172.804	-0.0044
L6U5	counter	218	35	-4.78	-4.78	84.00	-83.998	0.00202
L6U6	post	190	114	7.82	10.51	273.60	-273.6	-0.0003
U1U2	chord	107	199.5	2.66	-11.86	478.80	-478.801	-0.001
U2U3	chord	107	199.5	2.31	-23.85	478.80	-478.806	-0.0056
U3U4	chord	107	199.5	2.38	-32.85	478.80	-478.81	-0.0099
U4U5	chord	107	199.5	2.19	-39.52	478.80	-478.815	-0.0147
U5U6	chord	107	199.5	2.28	-43.26	478.80	-478.822	-0.0217

Table 7: Fixed-end forces, axial forces from nodal displacements, and effective axial forces from shrinkage in Model B

Nevertheless, it is apparent that the tangential and radial shrinkage at the nodes will, in actuality, affect the elements' connections and tend to reduce the prestress. These effects cannot, however, be analyzed with a model such as the ones used herein. A three-dimensional analysis using brick-elements would be needed to determine them.

For the creep analyses, two different procedures for the evaluation of creep nodal loading were used. One uses empirical data from the National Design Specification for Wood Construction, while the other employs a theoretical European method.

In the first, a value of strain equal to the initial elastic strain was used. The National Design Specification for Wood Construction suggests the value for the case of green wood in bending, and it has been confirmed by some empirical research.[42] It must be noted that most of the data used to evaluate a creep factor for the analysis of the Eldean Bridge were extrapolated from this research on creep in bending, instead of creep for axial loading, and on wood species different from white pine. The rationale for believing these data to be applicable is that, under steady moisture, creep deformation at a given percentage of the ultimate strength is roughly equal in compression, bending, and tension parallel to the grain, and that there is no marked difference in the creep behavior of wood among the different species.[43] For the second analysis, the creep strain was calculated using the equation given by the Eurocode 5.[44]

The effective nodal loads from creep (C_i) using the first procedure, were calculated for each element using the following formula:

$$C_i = \varepsilon_e \cdot E \cdot A_i$$

where ε_e is the initial elastic strain, E the modulus of elasticity of wood and A_i the area of the i-element.

In the second procedure, the effective nodal loads (C_i) were computed as follows:

$$C_i = \left\{ \frac{\sigma_i}{E} \cdot \left[1 + 0.30\left(1 + 0.30\left(1 - e^{-t}\right)\right)\right] \right\} \cdot E \cdot A_i$$

where σ_i is the applied constant stress in the elements and the time t (in hours) was taken equal to two years. This period of time factored in was two years, which available literature considers sufficient for creep to stabilize. Actually, this formula assumes a very high rate of creep, such that the values of strain after twenty-four hours were almost the same as those after two years.

After computing the axial forces from nodal displacements, the fixed-end forces were superposed on all the elements, and the actual axial forces due to creep were found. The

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[42] K.J. Fridley, "Designing for Creep in Wood Structures," *Forest Products Journal* 42, no. 3 (1992): pp.23-28.
[43] R.S.T. Kingston, "Creep, Relaxation and Failure of Wood," *Research Applied in Industry* 15, no. 4 (1962): pp.164-170.
[44] J.Tissaoui, "Effects of Long-Term Creep on the Integrity of Modern Wood Structures" (Ph.D. Diss., Virginia Polytechnic Institute and State University, December 1996).

values of the fixed-end forces, the axial forces from nodal displacements, and the effective axial forces due to creep are tabulated Table 8. Comparing the two methods, the creep nodal loads and the corresponding axial forces in the truss elements determined using the NDS empirical rule are 39 percent lower than those calculated by the Eurocode 5 formula.

Comparing axial forces from creep with axial forces from prestressing, it is evident that creep can loosen the prestressed counterbraces. To predict the times at which one or more counterbraces become loose, a model for a creep rate as function of stress is required. A qualitative "rule of thumb" for wood in a constant stress condition is that about 25 percent of the total creep occurs within the first day, 50 percent occurs within the first week and 75 percent occurs within the first month. As a general rule, wedges should be re-driven to compensate for this more often during the early stages of a bridge's life (within the first year) and less often later. Finally, the downward, mid-span displacements due to creep calculated in the two analyses performed, were of the same order of magnitude as the initial ones under dead load and prestressing (0.76 in). During experiments on the Eldean Bridge, about twenty wedges were re-driven.

Element		Dead load and prestressing		Creep (NDS)			Creep (EC5)		
Name	Type	Axial force prest. (kip)	Axial force dead & p. (kip)	Fixed end axial force (kip)	Axial force from nodal loading (kip)	Actual axial force (kip)	Fixed end axial force (kip)	Axial force from nodal loading (kip)	Actual axial force (kip)
L0L1	chord	0.03	14.35	-14.35	14.24	-0.11	-19.95	19.79	-0.16
L0U1	diagonal	-0.04	-29.05	29.05	-28.68	0.36	40.37	-39.87	0.51
L1L2	chord	2.70	28.67	-28.67	21.41	-7.26	-39.85	29.75	-10.09
L1U1	post	4.82	27.91	-27.91	14.50	-13.41	-38.79	20.15	-18.64
L1U2	diagonal	-5.31	-28.47	28.47	-13.54	14.94	39.58	-18.83	20.75
L2L3	chord	2.30	37.39	-37.39	31.78	-5.61	-51.97	44.17	-7.80
L2U1	counter	-5.57	-5.57	5.57	9.06	14.63	7.75	12.60	20.35
L2U2	post	8.74	27.28	-27.28	4.68	-22.60	-37.92	6.52	-31.40
L2U3	diagonal	-4.61	-22.71	22.71	-11.22	11.49	31.57	-15.60	15.97
L3L4	chord	2.39	44.00	-44.00	39.24	-4.76	-61.16	54.55	-6.61
L3U2	counter	-4.76	-4.76	4.76	6.47	11.23	6.62	9.00	15.61
L3U3	post	8.32	22.37	-22.37	4.03	-18.35	-31.10	5.60	-25.50
L3U4	diagonal	-4.76	-17.71	17.71	-8.02	9.68	24.61	-11.16	13.46
L4L5	chord	2.22	47.71	-47.71	43.93	-3.78	-66.32	61.07	-5.25
L4U3	counter	-4.92	-4.92	4.92	4.62	9.54	6.84	6.43	13.27
L4U4	post	8.10	17.60	-17.60	2.61	-14.99	-24.47	3.64	-20.83
L4U5	diagonal	-4.44	-12.12	12.12	-4.46	7.66	16.85	-6.20	10.65
L5L6	chord	2.16	48.96	-48.96	46.13	-2.84	-68.06	64.13	-3.93
L5U4	counter	-4.52	-4.52	4.52	3.01	7.53	6.28	4.19	10.47
L5U5	post	7.87	12.88	-12.88	1.32	-11.56	-17.90	1.84	-16.07
L5U6	diagonal	-4.26	-6.82	6.82	-1.17	5.65	9.48	-1.62	7.86
L6U5	counter	-4.78	-4.78	4.78	0.89	5.67	6.64	1.24	7.87
L6U6	post	7.82	10.51	-10.51	0.65	-9.86	-14.61	0.92	-13.69
U1U2	chord	2.66	-11.86	11.86	-19.05	-7.19	16.49	-26.49	-10.00
U2U3	chord	2.31	-23.85	23.85	-29.37	-5.53	33.14	-40.82	-7.68
U3U4	chord	2.38	-32.85	32.85	-37.52	-4.67	45.66	-52.15	-6.49
U4U5	chord	2.19	-39.52	39.52	-43.20	-3.67	54.94	-60.05	-5.11
U5U6	chord	2.28	-43.26	43.26	-45.98	-2.71	60.14	-63.92	-3.79

Table 8: Fixed-end forces, axial forces from nodal displacements, and effective axial forces from creep

Experimental Testing on the Eldean Bridge

Introduction

Considering the major feature of Long's truss, which is the precompression of the diagonal elements of the truss by driving in wedges, experiments on the Eldean Bridge were carried out with two main aims:

a) evaluating the probable prestressing load that can be applied to the counterbraces by manually driving the wedges against them and measuring the forces produced in the other members of a prestressed panel.

b) measuring the displacements of the bridge under a live load with loose wedges, and again after re-driving them, to compare the actual behavior of the bridge (counterbraces not prestressed) to the behavior of the original bridge (with prestressing) as conceived by Long.

The experimental results were also useful in the finite element modeling of the bridge. In particular, the measurements of the prestressing loads in the counterbraces were used to define those forces in the model. A comparison of the actual displacements obtained during the tests to those computed by the numerical model also served as a validity check of the model's accuracy as well as the assumed parameters.

Description of the Experiments

The battery of tests designed for the Eldean Bridge were performed over two days, June 12 - 13, 2002. Two basic kinds of tests were performed. One consisted of measuring strains in the members of selected panels in the north-east truss while driving the wedges against the counterbraces of those panels. The other tests involved measuring the displacements of the two bottom chords of the bridge at mid-span, for both the south and north spans, while driving a truck of known weight across the bridge. Table 9 lists the tests performed.

Kind of test	Test repetitions	Instruments	Performed analyses
Prestressing of counterbrace L3U2	twice	14 strain trans.	Strain-stress in the elements
Live load test on non-prestressed bridge	twice	14 strain trans. 3 displ. trans. 1 potentiometer	Mid-span displacements Strain-stress in the elements
Prestressing of counterbraces L3U2, L4U3, L5U4	Once, for three elements	14 strain trans.	Strain-stress in the elements
Truck of known weight traversing the bridge	twice	14 strain trans. 3 displ. trans. 1 potentiometer	Mid-span displacements Strain-stress in the elements

Table 9: Sequence of experimental tests performed on the Eldean Bridge

The strain test equipment consisted of fourteen strain transducers with a 5-inch gauge length, three inductive displacement transducers (DCDT), and a potentiometer (also used for displacement measurement). A multi-channel data logger (Fig. 22) was used for the collection and analog-digital conversion of the data. This was connected to a laptop computer that recorded the data and checked it for reasonableness. The sample interval was 1 per second in all tests. The data-logger and computer were time-synchronized prior to test initiation.

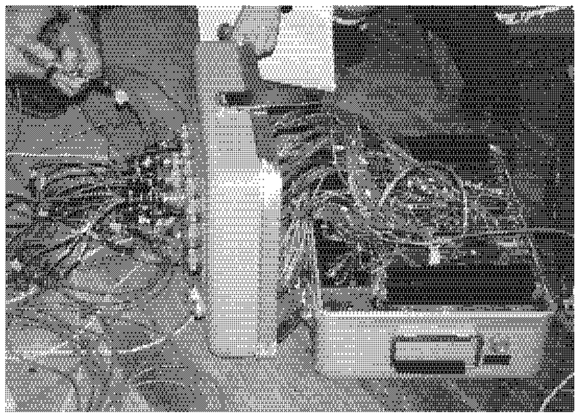


Fig. 22: Data logger. Field photograph, 2002.

Prior to the testing phase, a condition assessment of the bridge was conducted. The presence of loose wedges and the condition of the counterbrace joints were checked to confirm the expected lack of prestresses in the diagonal members of the truss. The few wedges that were found to still be tight were loosened in order to test the bridge with no prestressing whatsoever. A visual inspection of construction details, damage, and previous repairs was made in order to determine possible anomalies that might affect the test results. The inspection determined that the northeast truss would be the best on which to perform the strain, or "prestressing," tests. That truss contained a large number of loose wedges, but it was otherwise in good condition.

During the prestressing test on panel L2U2-L3U3, the loose existing wedge was re-driven into place with a sledge-hammer as described in Long's patent, in order to reproduce the original construction technique of the bridge as nearly as possible (Fig. 23). The strain produced by prestressing the counterbrace was measured in each member of the panel, using all fourteen available strain transducers. The counterbrace (one timber), main diagonal (two timbers), and posts (two timbers each) were instrumented with two strain transducers each (see Fig. 23). These were applied on the extreme fibers of the elements, in order to evaluate the average strain and detect possible bending on the members.

Stresses were calculated by multiplying the measured strains by the modulus of elasticity for white pine (1.2×10^6 psi). Also, the total axial forces were calculated by multiplying these unit stresses by the sectional areas of the appropriate members. This gave an indication of the possible prestress forces that can be produced in the counterbraces of Long trusses using the original prestressing technique. Following this test, the wedge was loosened in preparation for the un-prestressed live load test.

For the un-prestressed live load test, displacement transducers were installed at mid-span under all four bottom chords (see Fig. 24). Eight of the fourteen strain transducers (those on the posts) were removed from panel L2U2-L3U3 and installed on the main diagonal (L3U4) and counter (L4U3) of panel L3U3-L4U4, and on the counter (L5U4) of panel L4U4-L5U5.

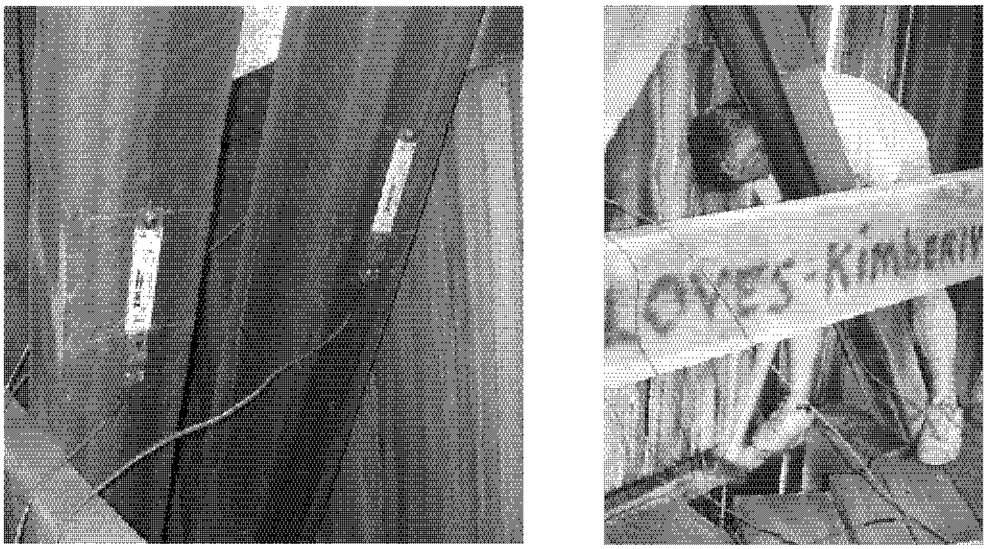


Fig.23: View of the main diagonal instrumented with strain gages on left; Driving a wedge against a counterbrace on right. Field photographs, 2002.

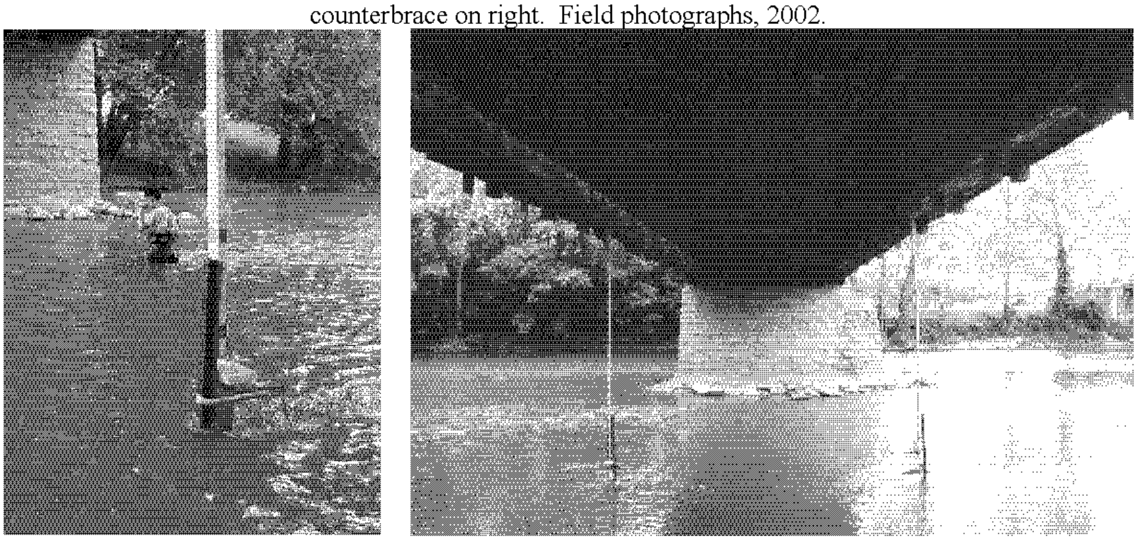


Fig. 24: Installation of a displacement transducer (left); View of the instrumented bridge (right). Field photographs, 2002.

The live load was applied by driving a truck weighing 7,900 pounds (3.95 tons) across the bridge (see Fig. 25). The truck was driven along the centerline of the deck to avoid uneven distribution of the load between the parallel trusses, and slowly to minimize dynamic effects. In each test the truck was moved from west to east, and it was stopped for about 10 seconds, to allow stabilization in the DCDT data acquisition, at six locations (west span mid-point [L6U6 post], east span L2U2 post, east span L4U4 post, east span mid-point [L6U6 post], east span L8U8 post, and east span L10U10 post).

Following the un-prestressed tests, all of the bridge's wedges were re-driven between the counters and the bottom chords to replicate the condition of the original, prestressed bridge as closely as possible. While prestressing Counters L3U2, L4U3, and L5U4 still had strain transducers installed, so the strains in those members were recorded during the prestressing activities, yielding a second set of "prestressing test" data for those counterbraces.

Finally, the truck was again driven across the bridge, in the same manner, to measure the bridge's behavior in its prestressed condition.

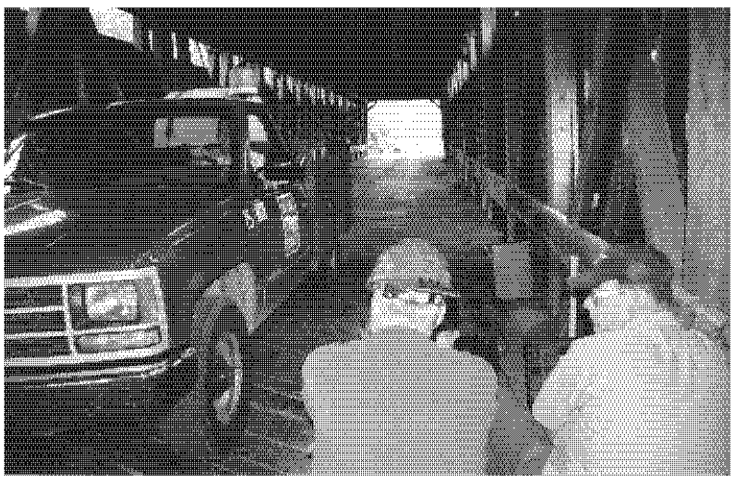
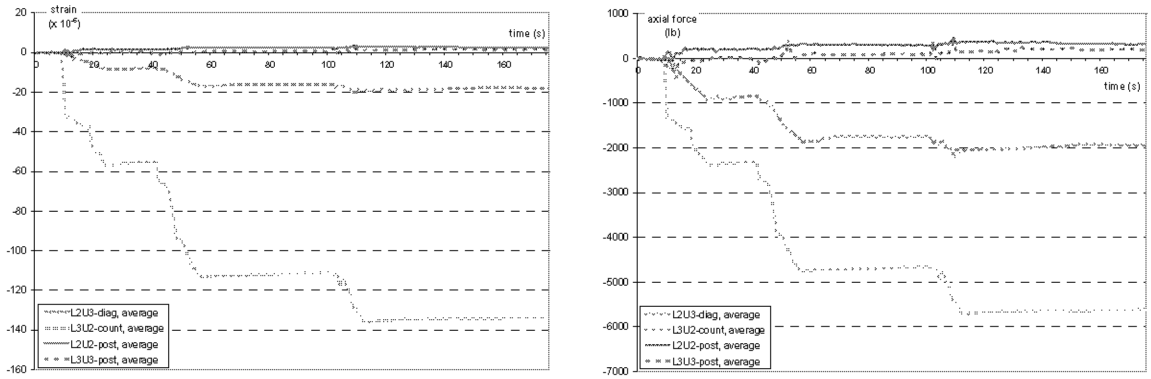


Fig. 25: Live load testing with the 7,900 lb truck. Field photograph, 2002.

Results of the "Prestressing Tests" on the Counterbraces

The "prestressing tests" lasted from 3 to 6 minutes. This was the range of time necessary to hammer each of the various wedges into position. The strains in the counterbraces, measured as averages of the various tests, were equal to -134.5 μstrain for L3U2 and -151.8 μstrain for L5U4, which correspond to average compressive stresses of -162 psi and -182.4 psi and average compressive axial forces equal to -5648 lb and -6375 lb. Prestressing forces were also produced in the main diagonals as well as in the posts of the tested panels. As expected, the main diagonals were subjected to an axial force of compression, while strains on the posts revealed the presence of low tension forces.



a) μstrain **b) axial force**

Fig. 26: Strains (a) and axial forces (b) produced in the elements of panel L2U2-L3U3 by prestressing the counterbrace

Results of the Live Load Tests

The curves plotted in figures 27 (un-prestressed) and 28 (prestressed) show the mid-span displacements of the four bottom chords as the truck moved across the bridge. The displacements are all plotted with respect to time, so they show how each mid-span point moved as the truck traversed the entire bridge. The slopes correspond to increases or decreases of the bottom chord mid-span displacements, while the truck is moving along the deck. The essentially horizontal portions of the curves are when the truck was stopped at various positions (west span mid-span, L2U2 east span, L4U4, east span mid-span, L8U8 and L10U10) for DCDT data stabilization. These flat areas of the curves correspond to static load conditions. The short-term variations are dynamic loads due to small bumps and the acceleration of the truck following the stops.

In the un-prestressed condition, the maximum average displacement of each span, with the load applied on that span, was about 0.083 inch (0.084 inch for the north-east chord, 0.081 inch for the south-east chord, 0.080 inch for the north-west chord, and 0.086 inch for the south-west chord). Not surprisingly, the southwest chord, which had shown the worst conditions during the visual inspection, experienced the greatest mid-span deflection.

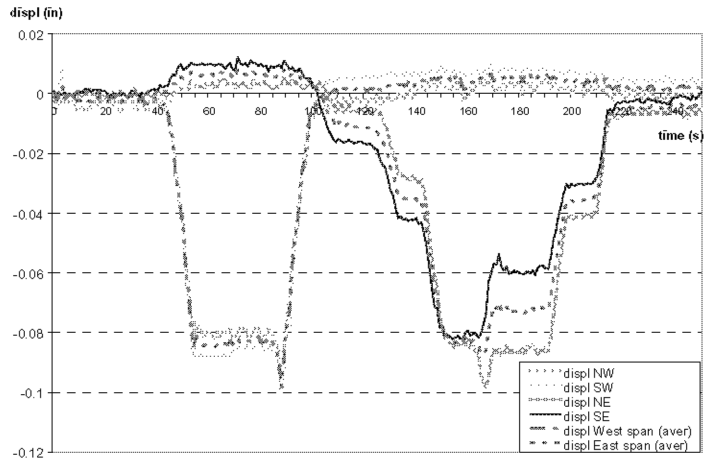


Fig. 27: Mid-span displacements under live load, with loose counterbraces

The average displacements of the two spans were the same, and the maximum deviation of a single chord displacement from the average displacement was ± 3.6 percent. The difference in displacements of the eastern span chords was probably due to some local damage in the southeast truss near mid-span.

In general in all the tests, when the load moved from the mid-span position L6, higher displacements were measured for positions L8 and L10 than for the corresponding L4 and L2 positions on the first half of the span. Also, after the bridge was completely unloaded, a residual displacement at mid-span, slowly decreasing with time, was observed. This clearly demonstrates how the stress-strain behavior of wood is time-dependent, or viscous. The behavior of the northeast span, where there was no change of mid-span displacement when the load moved from L6 to L8, could be related to the presence of broken shoulders on the posts.

Finally, despite the fact that the two spans of the bridge are separate and were expected to behave independently, a positive (upward) displacement of one span was observed when the load was applied to the other. The maximum amount of positive displacement measured was 0.004 inch for the western span and 0.006 inch for the eastern span. This interaction between the two bridge spans likely is caused by the roof and deck, both of which are continuous across the central pier. These displacements are small, equal to 5 percent and 7.5 percent of the maximum negative displacements reached in the western and eastern spans, respectively.

After prestressing the counterbraces of the eastern span, the measured displacements decreased by 12 percent, to an average for both chords of 0.073 inch. The wedges of the western span were not re-driven, so as expected, its average displacement remained practically the same as in the earlier tests.

Strains in selected members of the eastern span were also measured. Axial forces were evaluated by multiplying the measured strains by the modulus of elasticity for the wood

($1.2x10^6$ psi) and the section areas of the members. In the test with loose wedges, counterbraces were completely separated from the structure. Essentially zero strain and consequently stress and force were recorded for the duration of the test (see Fig. 29a). The two timbers that compose the main diagonal reached a maximum average negative strain of -28 μstrain, which corresponded to an average axial compressive force of -3,024 lb.

After re-driving the wedges the counterbraces became active. A positive 8.9 μstrain, which corresponded to an axial tensile force of 374 lb, was measured (see Fig. 29b). For the main diagonal the maximum strain and axial force, were -23 μstrain and –2,484 lb, respectively.

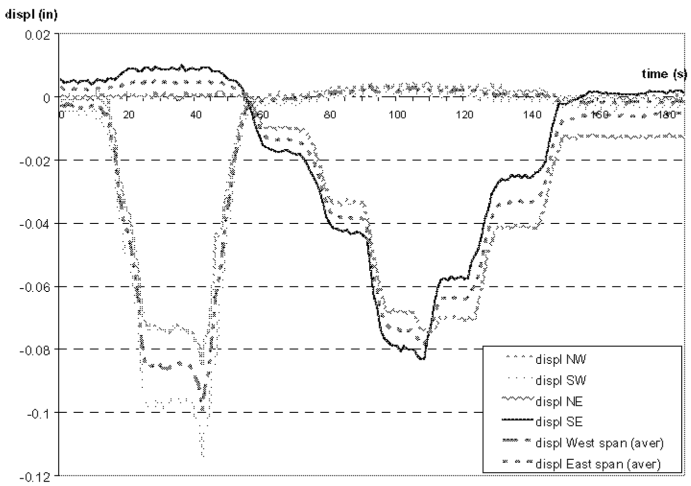
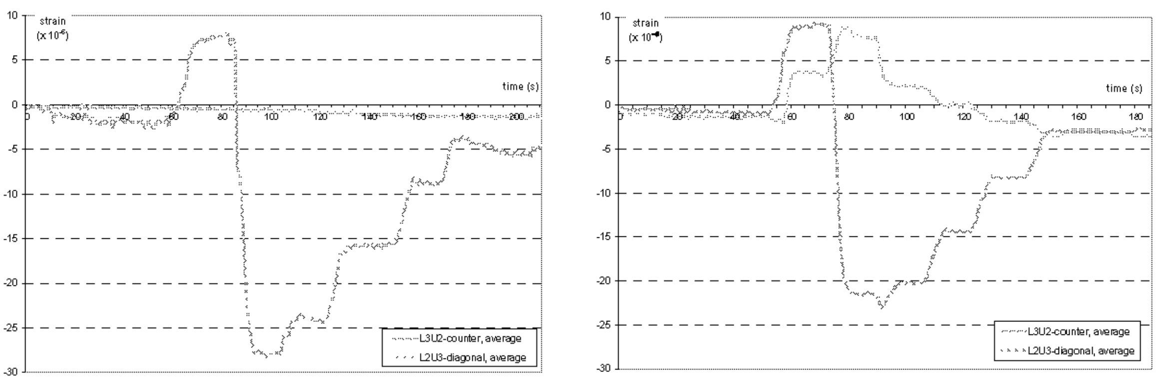


Fig. 28: Mid-span displacements under live load, with prestressed east span



a) un-prestressed bridge

b) prestressed bridge

Fig. 29: Average strains measured in the diagonal members of panel L2U2-L3U3 under live load, with loose counter (a) and prestressed counter (b)

These strain measurements reflect the effect of the live load only, because all the instruments were reset after re-driving the wedges. The strains produced in the diagonal elements by driving the wedges in panel L2U2-L3U3 had been previously measured during the "prestressing test." In the last prestressing test before the live load test they were equal, on average, to -16 and -97 μstrain, respectively, in the main diagonal L2U3 and counterbrace L3U2. These strains were added to those due to the action of live load in Fig. 30, in order to make an evaluation of the strain in the wooden members with the simultaneous action of prestressing and live load.

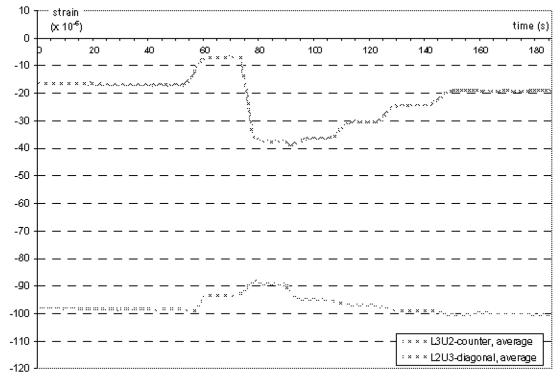


Fig. 30: Average strains in the diagonal members of panel L2U2-L3U3 under live load, including strains produced by prestressing the counters

As can be seen in Fig. 30, the negative strain in the main diagonal increases to a maximum average of -39 μstrain, which corresponds to an axial compressive force of –4,212 lb, when the live load is applied at L4. For the same load position, the strain in the counterbrace decreases from the -97 μstrain produced by the precompression, to -88 μstrain, which corresponds to an axial force of –3,696 lb. Therefore, during the live load application, the counterbrace did not reach a zero net axial force state, but remained in compression. Because of the prestressing, the counter did not separate from the structure and, thus, contributed significantly to the stiffness of the bridge.

Fig. 31 shows the evolution of average strain in the various elements instrumented with strain transducers as the live load traversed the bridge. In this diagram the previous strains due to the prestressing of the counterbraces have not been considered. The necessity of proper prestressing to keep the counterbraces in compression under live loads is again clear.

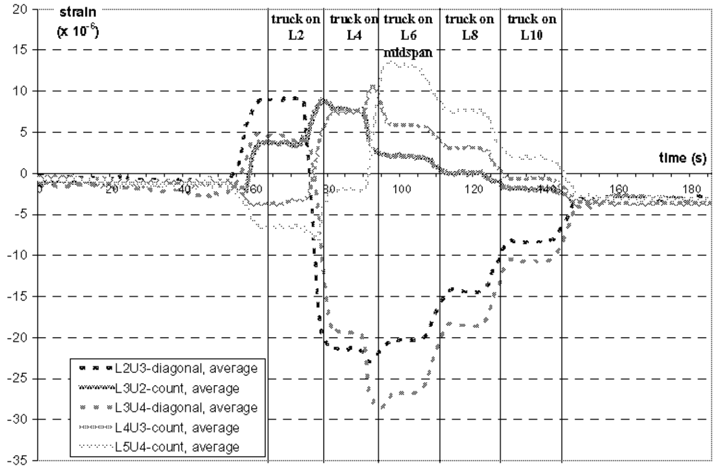


Fig. 31: Average strains in the diagonal elements of the truss under live load, with prestressed east span (strains due to prestressing not included)

Comparison Between Experimental Testing and Numerical Modeling

The combination of theoretical and empirical studies of the Eldean Bridge presented several opportunities to compare theoretically predicted performance values with those measured in the field. The two most important of these parameters are displacement (deflection) and axial forces in key members.

The mid-span location was selected as the point for displacement comparison. With a 3.95 kip load applied at the east mid-span point, the average displacements measured for the bridge with un-prestressed (loose) and prestressed counters were equal to 0.083 inch and 0.073 inch, respectively. The results of the structural analyses, for the same conditions, were, respectively, 0.089 inch and 0.066 inch. Interestingly, the model predicted a 7 percent greater deflection than measured for the un-prestressed bridge, but 9 percent less than measured for the prestressed span. Fig. 32 shows this comparison. While the overall comparison is reasonably good, note that the numerical results are more symmetrical about the mid-point than the values measured in the field. As discussed above, dynamic effects, differences in member condition, and the limited interaction between spans are probable contributors to the difference.

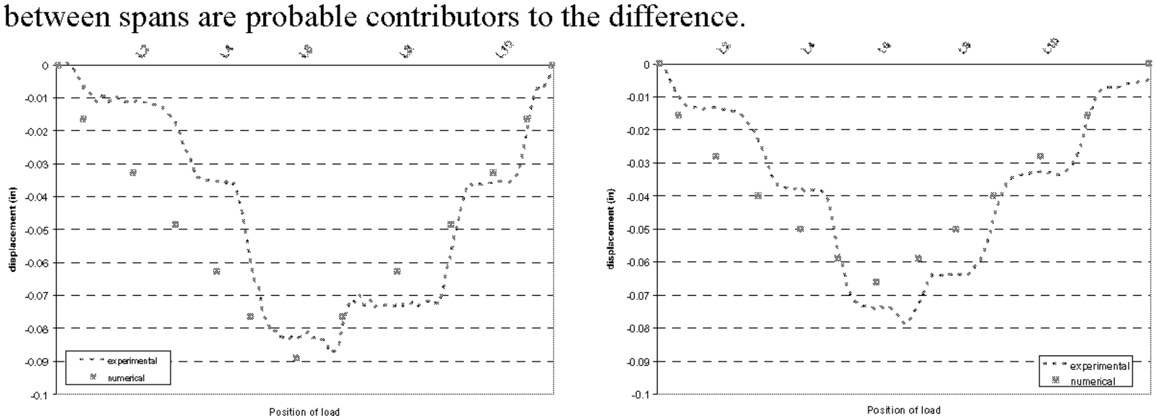
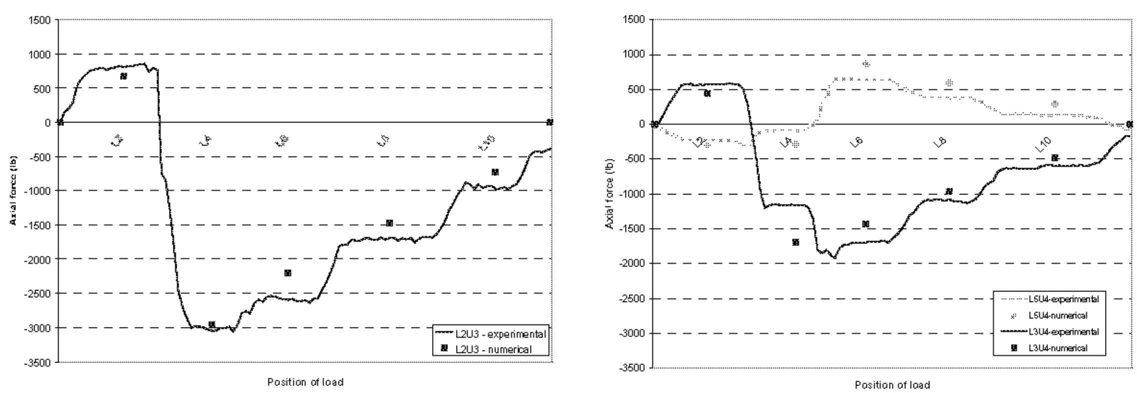


Fig. 32: Comparison between measured displacement at mid-span with different loading position and numerical results; un-prestressed (left) and prestressed (right)

A comparison between the maximum axial forces experimentally determined (calculated using the measurement of strain) and the corresponding axial forces predicted by the model is shown in Fig. 33. Again, results for un-prestressed and prestressed conditions are included.

In the case of the un-prestressed truss (a), the maximum experimental axial forces (in diagonal L2U3) were 811 lb. and -3046 lb., when the live load was applied at L2 and at L4, respectively. The corresponding numerically generated axial forces were 671 lb. and -2966 lb. This is fairly good to very good agreement. As with nodal displacements, a residual axial force was measured after unloading the bridge, something that could not be predicted by the linear elastic numerical analyses.



a) b)

Fig. 33: Comparison between experimentally measured axial forces in the elements and numerical results for un-prestressed (a) and prestressed (b) bridge

In the prestressed truss (Fig. 33b), the maximum experimental axial forces in the diagonal L3U4 were equal to 570 lb. and -1203 lb., with the live load applied at L2 and at L4, respectively. The corresponding numerically generated axial forces were 447 lb and – 1,690 lb, differences of 22 and 29 percent. In counterbrace L5U4 the experimental values were -93 lb and 641 lb, with the load applied at L4 and L6, while the numerical values were -285 lb and 864 lb.

While Fig. 33 shows general agreement between the experimental and numerical influence lines, there are some anomalies. For example, considering the L3U4 influence lines, the ordinate for load position L6 should be smaller than for load position L4. Similarly, in Fig. 31, the L3U2 influence line ordinate for load position L2 should be negative (though the positive value measured was very small, about 4 μstrain). These discrepancies may have occurred because the centroid of the truck was only visually estimated and because the stopping points were not controlled precisely.

There were discrepancies between the calculated and measured values, but in general, the numerical models used described the short-term behavior of both the un-prestressed and prestressed truss reasonably well.

Conclusions

Observations on the Static Behavior of the Eldean Bridge

The numerical studies of the static behavior of Eldean Bridge showed that the maximum axial stresses under dead load in the elements were less than 300 psi. Also, stresses from bending moment were less than one-tenth of those from axial forces, and shear forces in the member are almost zero.

The maximum vertical displacements of the Eldean Bridge under dead load were predicted to be 0.64 inches without the effect of the counterbraces, and 0.54 inches with the action of counterbraces. The analyses thus attributed a 16 percent increase of the bridge's stiffness to active counters, indicating that Long's prestressing technique is effective.

The action of driving in (tightening) a wedge primarily affects only one panel, therefore, all of the counterbraces must be prestressed to assure a uniform behavior of the bridge. Prestressing does not cause an upward displacement of the truss, therefore, it does not relieve any of the dead load forces from falsework used to construct the bridge, nor does it add to the bridge's camber. Moreover, unless dead load displacements would be excessive, it is, as Long recommended, sensible to prestress a truss with the falsework removed and the dead load already active, to avoid an immediate decrease of prestress in the counters once the falsework is removed.

With a uniformly distributed live load of 40 lb/ft^2 on the entire Eldean Bridge, the maximum compressive stress due to live and dead load was 434 psi, a value that is reasonably less (36 percent) than the maximum allowable stresses found in the National Design Specification for Wood Construction. It is not, however, possible to make a statement in this report on the reliability of the bridge, because its strength could be limited to a lesser load by the capacity of the joints, and the joint quality was not evaluated.

The minimum concentrated live load that would cause slackness in any element of this truss is 5.74 tons, which would loosen the central main diagonals if the counters were inactive. About 14 tons would loosen the counterbraces of the panels adjacent to the end ones if the counters were active. These values demonstrate the improved behavior of the bridge with the counters prestressed as intended. The same two elements are loosened, in the two bridge conditions, for uniformly distributed live loads of 38 lb/ft^2 and 46 lb/ft^2 on one-half span of the bridge. These critical uniform live loads are less than the design loads used for rural bridge design by the end of the nineteenth century, although earlier builders may have used lower design loads.

Longitudinal wood shrinkage did not cause significant stresses in the elements, nor a loss of prestress in the counterbraces. This is because the shrinkage was uniform for all truss elements, and because the truss was free to shrink as a simply supported structure. This would not have been true if different elements had different shrinkage properties, or if free shrinkage of the truss had been prevented. Shrinkage also caused downward displacements of very small magnitudes. The effect of tangential and radial shrinkage at nodes was not evaluated. This would have required a three-dimensional model that was not available.

It was possible to determine that creep caused a loss of prestress and, possibly, slackness in the counterbraces. This occurred because creep strains were not equal, since the dead and live load stresses were different in each element. The numerical analysis indicated that the first critical counters to have become slack were those closest to the span ends. It was not possible to predict the time to slackness, since a viscous stress-strain model for wood that includes the changes in creep stresses with time would be needed.

The shrinkage and creep analyses reported here are useful only to give a qualitative description of the phenomena. They provide a general idea whether slackness will occur and where, but they cannot give more detailed information on joint effects or on time to slackness. It is evident that shrinkage and creep do affect the structural behavior of a Long truss bridge, but periodic re-tightening of wedges, especially during the early stages of the bridge's life, may be sufficient to maintain the bridge in proper condition.

Observations on the Experiments and Comparison with Numerical Analyses

With a simple test, it was possible to evaluate the axial forces produced by driving wedges, the technique proposed by Long, to prestress the bridge. With this simple evaluation, the order of magnitude of the prestressing load that can be achieved in the counterbraces is about 5 - 6 kips, and it is possible to carry out structural analyses that properly take into account the prestressed state of the bridge.

The comparison of live load test results before and after prestressing the counterbraces, allowed quantification of the effectiveness of re-driving the wedges in stiffening the bridge. Active counters reduced the mid-span displacements and redistributed axial forces in the diagonal elements of the truss. The additional stiffness from prestressing the counters, measured in terms of reduction of mid-span displacement under live load, was equal to about 12 percent.

Experimental influence lines for certain element forces and vertical displacements were plotted, and these allowed interpretation of the actual behavior of the bridge. For example, the influence lines for vertical displacements at mid-span revealed a viscous effect in the bridge response under live load.

Finally, the reliability of the numerical model used for the structural analysis was assessed. An overall comparison of the maximum displacements under live load, as experimentally measured and numerically calculated, revealed good correlation. The actual stiffness of the bridge was properly evaluated by numerically modeling the bridge with both loose and prestressed counterbraces.

In the case of the Model A, representing the bridge with the loose counterbraces, a good agreement with the experimental results also was found for the axial forces in the elements. In the case of the Model B, the agreement in terms of axial force in the element was not as good, but the model still was able to capture the overall behavior of the bridge. These numerical models could not reveal effects due to local damage.

While they have certain limitations that arise from some simplifying assumptions about joint condition, wood behavior over time, and local damage or deterioration, these linear elastic plane frame models are adequate to predict short-term behavior of wooden trusses—the Eldean Bridge truss in particular—with useful accuracy.

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